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INFORMATION RETRIEVAL

The key words, abstract, and reference "cards" for each article in this Journal represent part of the ASCE participation in the EJC information retrieval plan. The retrieval data are placed herein so that each can be cut out, placed on a 3 × 5 card and given an accession number for the user's file. The accession number is then entered on key word cards so that the user can subsequently match key words to choose the articles he wishes. Details of this program were given in an August, 1962 article in CIVIL ENGINEERING, reprints of which are available on request to ASCE headquarters.

*Discussion period closed for this paper. Any other discussion received during this discussion period will be published in subsequent Journals.

16558 HYDROLOGIC MODEL FOR DROUGHT MANAGEMENT

KEY WORDS: Distribution functions; **Droughts; Forecasting; Hydrologic data; Probability theory;** Risk; **Rivers;** Stochastic models; **Streamflow forecasting;** Time factors; Water flow

ABSTRACT: A method of producing long-range water-supply forecasts is developed and applied. The method is capable of producing conditional cumulative distribution functions (CDF) for monthly streamflow volumes and producing a set of equiprobable future streamflow traces. Using the latter output, it is possible to evaluate the conditional CDF of reservoir storage for any particular future month, or probabilities of storage falling below some level during the forecast period. The model used in an ARMA (1,1) cyclic model. It differs from an ARMA (1,1) stationary model in that the correlation structure varies with time of year. The model is fit in this analysis to the 81-year long stream-flow record of the Potomac River at Point of Rocks, Maryland.

REFERENCE: Hirsch, Robert M. (Hydro., Systems Analysis Group, Water Resources Division, U.S. Geological Survey, National Center, Reston, Va. 22092), "Stochastic Hydrologic Model for Drought Management," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16558,** October, 1981, pp. 303-313

16578 WETLANDS' USE FOR WATER MANAGEMENT IN FLORIDA

KEY WORDS: Drainage; Flood control; **Florida; Freshwater marshes;** Hydrology; Phosphorus; **Water management (applied);** Water quality; Water supply; **Wetlands**

ABSTRACT: The 2,300 square mile (6,000 km^2) Kissimmee River Basin in Central Florida is under pressure for rapid expansion due to urban encroachment and agricultural activities. The river contributes the major portion of surface inflow and a substantial portion of the nutrient loads to Lake Okeechobee, the primary water supply source for South Florida. A basin-wide water management scheme has been proposed based on detention of runoff, restrictions of surface water discharge rates, and routing flow through natural or manmade marshes. The ability of these marshes to serve as quantity and quality control areas is evaluated, and specific questions are addressed concerning trade-offs among drainage, flood control, water supply, and water quality enhancement.

REFERENCE: Ammon, Douglas C. (Staff Engr., EPA, Cincinnati, Ohio 45268), Huber, Wayne C., and Heaney, James P., "Wetlands' Use for Water Management in Florida," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16578,** October, 1981, pp. 315-327

16562 LOCAL IRRIGATION AGENCIES

KEY WORDS: Conjunctive use; Ground water; **Irrigation;** Irrigation engineering; Irrigation systems; **Local governments;** Public services; Surface waters; **Water distribution;** Water management (applied)

ABSTRACT: Local irrigation agencies are not centralized institutions with complete power over all surface and groundwater decisions in their basins. Instead, the three major types of irrigation agencies (public irrigation districts, mutual water companies and commercial water companies) have different and relatively limited power. Most importantly, with certain exceptions, they cannot directly control excessive groundwater withdrawals and have to resort to indirect methods to achieve this goal. Thus, the phrase "Conjunctive management of surface and groundwater" is very misleading. Conjunctive use "models" should only include in their proposed objective functions those decision variables controlled by the particular agency under study.

REFERENCE: Revesz, Richard L. (Former Grad. Research Asst., Mass. Inst. of Technology, Cambridge, Mass.), and Marks, David H., "Local Irrigation Agencies," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16562,** October, 1981, pp. 329-338

16574 SAM RAYBURM RESERVOIR

KEY WORDS: Models; **Multiple purpose reservoirs;** Optimization; Planning; Reservoirs; Simulation; Systems analysis; **Texas; Water resources**

ABSTRACT: A combined optimization-simulation procedure was developed and applied to identif the optimal conservation pool storage allocation problem is formulated and solved as a constrained nonlinear programming (NLP) problem with multiple objectives. The Box-Complex algorithm is coupled with an existing generalized reservoir simulation model to seek a solution to the NLP problem. The solutions are refined by an iterative simulation process that allows input from planners and engineers who are involved in management of this system.

REFERENCE: Ford, David T. (Hydr. Engr., U.S. Army Corps of Engrs., The Hydrologic Engrg. Center, 609 Second St., Davis, Calif.), Garland, Ralph, and Sullivan, Charles, "Operation Policy Analysis: Sam Rayburn Reservoir," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107,. No. WR2, **Proc. Paper 16574**, October, 1981, pp. 339-350

16572 INTERACTIVE NONSTRUCTURAL FLOOD-CONTROL PLANNING

KEY WORDS: Benefit cost analysis; Community planning; **Computer programs; Data retrieval;** Data storage; Economic analysis; **Flood control;** Flood plains; **Formulation; Information systems**

ABSTRACT: The partical success of any flood control plan depends on the ability of the planner to analyze the available data to formulate a feasible plan, considering systematically the many possible alternative solutions. Computer programs of the Nonstructural Analysis Package, developed in the Hydrologic Engineering Center, obviate the data storage, manipulation, and retrieval problems and provide for rapid evaluation of alternative nonstructural flood-control plans. The package consists of two programs: (1) The preprocessor program that digests hydraulic, hydrologic, economic, and engineering information for each structure of group of structures in a flood plain and prepares a random access data bank; and (2) the interactive analysis program that selectively accesses, displays and manipulates the data. the program package had beem employed successfully in development of a proposed nonstructuraflood-control plan for Santa Fe, New Mexico.

REFERENCE: Ford, David T. (Hydr. Engr., The Hydrologic Engineering Center, U.S. Army Corps of Engrs., Davis, Calif. 95616), "Interactive Nonstructural Flood-Control Planning," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16572**, October, 1981, pp. 351-363

16565 ENERGY COSTS AND PORTLAND WATER SUPPLY SYSTEM

KEY WORDS: Costs; Electric power; **Electric power costs; Energy conservation; Municipal water;** Operation and maintenance; Pumps; Urban areas; **Water management (applied);** Water plans; Water resources

ABSTRACT: The changing role of electrical energy on the Portland, Oregon, municipal water supply system is presented. Portland's actions in energy conservation include improved operating procedures, pump modifications and modifications to the water system to eliminate pumping. Portland is implementing a "small hydroelectric" project at exisiting water supply dams to produce an additional source of power for the area. Special precautions in construction and operation are necessary to protect the high quality of the water supply.

REFERENCE: Elliott, William M. (Water Utility Engr., Bureau of Water Works, 1800 S.W. Sixth Ave., Portland, Oreg. 97201), and Hawley, Randy P., "Energy Costs and Portland Water Supply System," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16565**, October, 1981, pp. 365-374

16576 ACCESS TO WATER-RELATED DATA IN TEXAS

KEY WORDS: Computers; Data banks; Data collection; Government agencies; Information centers; **Information retrieval; Information systems; Texas; Water consumption; Water resources; Water use**

ABSTRACT: The TNRIS (Texas Natural Resources Information System) is a mechanism in the Texas state government which links the users of naturaal resources and related data with those organizations which collect and store the data. The system is an extension of work to develop a Texas Water Oriented Data Bank which began in 1967. TNRIS provides for access to data in six major categories: hydrological, biological, meterological, geological, socio-economic, and base data. Users are in government, education and the the private sector. Data indexing, on-line data access, interface with other systems, and development of computer software to manipulate and display natural resource data are areas of major TNRIS involvement. More than 450 data and information files are available through TNRIS.

REFERENCE: Ferguson, David L. (Asst. Dir., Admin. Serv. Div., Tex. Dept. of Water Resources, Austin, Texas 78711), "Access to Water-Related Data in Texas," *Journal of the Water Resources Planning and Management Division*, ASCE, Vol. 107, No. WR2, **Proc. Paper 16576**, October, 1981, pp. 375-383

16560 DETENTION BASIN PLANNING AND DESIGN MODELS

KEY WORDS: Comparative studies; **Design criteria;** Detention reservoirs; **Drainage; Drainage basins; Flood control; Models; Runoff;** Storm water; Water management (applied)

ABSTRACT: A basis is provided for selecting a suitable stormwater detention basin design method by identifying the various solution methodologies and by comparing the results. Two design methods, which included hydrologic and hydraulic routing elements, and five planning methods that included only watershed hydrology were applied to watersheds of 50 acres, 200 acres, and 500 acres, and for residential, commercial and industrial land uses. Return periods of 10-yr and 100-yr were used. Based on consideration of model characteristics, one design and one planning method were recommended for providing consistently acceptable results.

REFERENCE: Donahue, John R. (Grad. Research Asst., Dept. of Civ. Engrg., Univ. of Maryland, College Park, Md. 20742), McCuen, Richard H., and Bondelid, Timothy R., "Comparison of Detention Basin Planning and Design Models," *Journal of the Water Resources Planning and Management Division*, ASCE, Vol. 107, No. WR2, **Proc. Paper 16560**, October, 1981, pp. 385-400

16559 IN-HOME CONSERVATION AND WASTEWATER MANAGEMENT

KEY WORDS: Construction costs; Cost effectiveness; Flow rates; **Grants; Mathematical models; Municipal water; Wastewater treatment; Water conservation;** Water consumption; Water management (applied)

ABSTRACT: A mathematical model has been developed to evaluate the cost-effectiveness of household water conservation practices as a component in municipal wastewater treatment to meet federal secondary treatment requirements. The analysis uses a net present equivalent method and considers factors such as: time-value of money, inflation, population growth rate, public acceptance of household water conservation practices, residential contribution to the treatment plant influent, and both construction and operations and maintenance costs for municipal wastewater treatment plants. A sensitivity analysis is also included. Results show that while household water conservation practices do provide a cost-effective component of municipal wastewater treatment under a wide range of reasonable conditions, the municipal construction grants program may provide a disincentive.

REFERENCE: Hopp, Wallace J. (Research Scientist, Battelle Pacific Northwest Labs. Richland, Wash.), and Darby, William P., "In-Home Conservation and Wastewater Management," *Journal of the Water Resources Planning and Management Division*, ASCE, Vol. 107, No. WR2, **Proc. Paper 16559**, October, 1981, pp. 401-418

16557 RAINFALL MONITORING-PREDICTION SYSTEM

KEY WORDS: **Chicago;** Flooding; **Forecasting;** Hydrology; **Meteorology;** Monitoring; Radar; **Rainfall;** Rainfall-runoff relationships; Remote sensing; Storm sewers; **Storm water; Urban areas**

ABSTRACT: During a 4-year period (1975-1979), a comprehensive hydrometeorological research program was carried out in the Chicago metropolitan area to develop better methods of utilizing meterorological data in the design and real-time operation of urban hydrological systems. A real-time monitoring-prediction rainfall system was developed and tested to facilitate and improve operations of urban storm-sanitary sewer systems during heavy rainstorms. Basically, this system involved the combination of a dedicated weather radar with associated data processors and computers, used in conjunction with telemetered raingage data. The demonstration proved successful and indicated that the system can provide very useful meteorological input which should lead to substantial improvement in the real-time operation of urban hydrologic systems, particularly in large metropolitan areas having complex operating problems during heavy rainstorms.

REFERENCE: Huff, Floyd A. (Meteorological Consultant, Pembroke Pines, Fla.), Vogel, John L., and Changnon, Stanley A., Jr., "Real-Time Rainfall Monitoring-Prediction System," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16557,** October, 1981, pp. 419-435

16561 ANALYTICAL COST MODEL FOR URBAN WATER SUPPLY

KEY WORDS: **Cost effectiveness; Economic analysis;** Economics; Planning; **Regional planning; Urban areas;** Urban planning; **Water consumption; Water distribution; Water supplies;** Water transportation; Water treatment

ABSTRACT: The local water utility is ordinarily a pure monopoly except that industrial customers of water may establish their own supplies. Water supply systems can be separated into components: (1)Acquisition-treatment; and (2)delivery. Each of these components has its own cost function. Economies of scale gained in treatment may be lost in transportation. The tradeoffs between economies of scale in production and the costs of transportation have significance for the development of regional water supply systems. Cost functions are developed that allow a decision maker to evaluate these tradeoffs. Under specific circumstances the minimum average cost point for delivered water is approximately 8 miles.

REFERENCE: Clark, Robert M. (Engrg. Systems Analyst, Drinking Water Research Division, MERL-EPA, Cincinnati, Ohio 45268), and Stevie, Richard G., "Analytical Cost Model for Urban Water Supply," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16561,** October, 1981, pp. 437-452

16589 SALINITY FORUM: WHAT? HOW? WHY?

KEY WORDS: **Colorado River;** Government agencies; **Regional planning;** River basins; **River systems;** Saline water; **Salinity;** Water management (applied); Water pollution; **Water resources**

ABSTRACT: Historically, salinity in the Colorado River System has increased significantly due to export of high quality water from the upper reaches and salt pickup from irrigation return flow. Actions in any part of the basin (including parts of Arizona, California, Colorado, Nevada, New Mexico, Utah, and Wyoming) affect the problem, and a basinwide approach is highly desirable. Although damages due to salinity are principally experienced in the lower basin, implications of the Clean Water Act make solving the problem a matter of concern for the upper basin as well. The Colorado River Basin Salinity Control Forum, an interstate organization, was created in 1972 to address the problem. The forum has been very successful because there is a clear mission to which all members are committed, and the forum members can normally expect their decision to be supported by their governors.

REFERENCE: Lawrence, Daniel F. (Dir., Utah Division of Water Resources Salt Lake City, Utah), and Saunders, Barry C., "Salinity Forum: What? How? Why?," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16589,** October, 1981, pp. 453-459

16538 DEVELOPING DAM-BREAK FLOOD ZONE ORDINANCE

KEY WORDS: **Dam failure; Dams;** Dam stability; Flood control; **Floods; Georgia;** Guidelines; **Land use; Legislation;** Models; **Urban planning; Zoning**

ABSTRACT: An interagency task force has been assembled to work with Gwinnett County, Georgia, on a dam-break flood zone ordinance and to develop a model ordinance for Georgia counties and municipalities. The purpose of the model ordinance is to control development downstream of low-hazard dams so that these dams remain in a low-hazard category. Guidelines developed to assist state agencies in determining dam-break flood zones are presented. The guidelines assemble a great deal of data and theoretical developments applicable to simulating dam-break floods. The level of effort required in delineating dam-break flood zones in Gwinnet County is discussed, and the model ordinance is presented in its entirety.

REFERENCE: McMahon, George F. (Hydr. Engr., U.S. Army Corps of Engineers, Savannah Dist., Savannah, Ga.), "Developing Dam-Break Flood Zone Ordinance," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16538,** October, 1981, pp. 461-476

16584 WATER QUALITY MODEL FOR A RIVER NETWORK

KEY WORDS: Backwater; Channels (waterways); Computers; Deltas; **Hydraulics; Mathematical models; River engineering;** Rivers; Tides; **Time dependence;** Waste disposal; Water pollution; **Water quality**

ABSTRACT: A complex river network system consisting of branches and loops is sometimes complicated by downstream effects from tides, lakes, and other factors. The management of water quality in such rivers is a difficult task. A model was developed to simulate hydraulic behavior and water quality of a river network on a one-dimensional representation. The two complete St. Venant equations and the transport equation were solved by a finite difference implicit method using a modified Gauss elimination procedure. The model can simulate biochemical oxygen demand, dissolved oxygen, and any conservative substance. The hydraulic module of the model was adjusted and verified with data from the Jacui Delta, Brazil, showing good agreement between the calculated results and the observed data. The water quality model was tested under hypothetical conditions for the same Delta to demonstrate the utility of the model in making managerial decisions.

REFERENCE: Tucci, Carlos E.M. (Assoc. Prof., Instituto de Pesquisas Hydraulicas, Porto Alegre, RS, Brazil.), and Chen, Yung Hai, "Unsteady Water Quality Model for River Network," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16584,** October, 1981, pp. 477-493

16579 MULTIPURPOSE TVA RESERVOIR MODEL

KEY WORDS: Cost effectiveness; **Dynamic programming;** Flood damage; Hydroelectric power; **Mathematical models; Operating costs;** Optimization; Recreation; **Reservoir engineering;** Reservoir operation; **Reservoirs;** Reservoir storage; Systems engineering; **Tennessee Valley Authority;** Water plans; Water quality; Water resources; **Water resources management;** Water supply systems

ABSTRACT: A mathematical model is described that was developed as a tool for reservoir system planning studies and long-range operational studies. It consists of a simulation model of the reservoir system guided by an optimization method based on dynamic programing. The objective function represents total system operation cost. This function consists of the weighted sum of cost functions representing the major operating purposes of the system. The model determines end-of-week storage levels for 52 weeks or shorter planning periods for 19 storage reservoirs. The optimal solution is associated with minimum system operation cost for the planning period. The model is deterministic. It uses a set of power-system data and hydrologic data and finds the least cost solution for this input by considering the entire planning period. Specific operating constraints can be included.

REFERENCE: Giles, James E. (Mathematician, TVA Water Management Methods Staff, Norris, Tenn.), and Wunderlich, Walter O., "Weekly Multipurpose Planning Model for TVA Reservoir System," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16579,** October, 1981, pp. 495-511

16568 OPTIMAL STREAM-AQUIFER DEVELOPMENT

KEY WORDS: Aquifers; **Conjunctive use; Decomposition; Design;** Dynamic programming; Hydroelectric power; Irrigation; **Mathematical models;** Operation and maintenance costs; **Optimization;** Reservoirs; **Water resources**

ABSTRACT: A mathematical programming model is presented for a conjunctive development problem of surface and ground water resources. The surface water is to be used for power generation, irrigation, and aquifer recharge purposes, while meeting the mandatory downstream requirements. A confined aquifer is to be used for irrigation purpose only. Design and operation decisions are considered simultaneously. Design of the system includes determination of the sizes of dam, powerplant, canal, and ground water pumps. Operation of the system determines the release policy for each purpose from each source. The resulting model is a large scale nonconvex problem.

REFERENCE: Coskunoglu, Osman (Asst. Prof, Dept. of General Engrg., Univ. of Illinois, Urbana, Ill.), and Shetty, C. M., "Optimal Stream-Aquifer Development," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16568,** October, 1981, pp. 513-531

16548 COMPUTER MODELLING OF FLOOD ALLEVIATION BENEFITS

KEY WORDS: Computer applications; **Computer models;** Cost effectiveness; **Data banks;** Data collection; **Flood control; Flood damage;** Flood hydrology; **Floods; Sensitivity analysis; United Kingdom;** Urban areas

ABSTRACT: New data and techniques have been developed in the United Kingdom to simplify economic evaluations of flood alleviation schemes. Data banks have been developed containing standardized flood damage data applicable to any British flood situation for any type of property. The research responds to recommendations by the British Institution of Civil Engineers in 1967 that flood damage information should be systematically collected. The computer model is explored that was developed to use this flood damage data when calculating the discounted annual benefits for urban alleviation schemes. The model and associated data banks are used by regional water authorities in appraising capital expenditure. Although developed for application in UK, with a change of data base the model could be used in any flood context.

REFERENCE: Chatterton, John B. (Co-ordinator Rivers and Directorate of Operations, Severn-Trent Water Authority, Sheldon, Birmingham B26 3PU, England), and Penning-Rowsell, Edmund C., "Computer Modelling of Flood Alleviation Benefits," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16548,** October, 1981, pp. 533-547

16608 DECISIONS AND REGRESSION NORMALITY

KEY WORDS: Culverts; Hydrology; **Linear regression analysis;** Management; Management planning; Normality; Runoff; **Statistical analysis; Water resources management**

ABSTRACT: Two water resources engineering questions are considered to study the statistical effect and associated loss of incorrectly assuming normality in linear regression analysis. Monte Carlo simulation is used to investigate the small sample behavior of the estimated regression coefficients, estimated variances, and predicted values of the dependent variable when the errors are not normally distributed. The distributions simulated are lognormal, Pearson and LogPearson with varying degress of skew. The estimated values of the regression coefficients are not appreciably affected by skew of the error distribution. Decisions based on both the mean and variance are less robust to nonnormality than decisions based only on the mean. Many hydrologic decisions are based on confidence intervals or percentiles, which involve estimated standard deviations and, thus, may be quite sensitive to violation of the normality assumption.

REFERENCE: Castano, Eugenio (Former Doctoral Canadidate, Univ. of Arizona, Tucson, Ariz. 85721), Duckstein, Lucien, and Weber, Jean, "Decisions Under Violation of Regression Normality," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16608,** October, 1981, pp. 549-561

16575 MANAGEMENT OF INTERSTATE AQUIFER SYSTEMS

KEY WORDS: **Aquifers;** Federal government; Federal laws; **Ground water; Institutions; Interstate commissions; Management;** State governments; **State laws;** Water law; Water resources management

ABSTRACT: Designing a "workable" institutional structure for comprehensive management of an interstate aquifer system involves consideration of not only the geology and hydrology of the system, but also other factors. These include the differing state water laws, present intrastate institutional structures for management, exisiting patterns of ground-water use, Federal and state water programs, and water user attitudes. Comprehensive management of the Ogallala Aquifer (underlying West Texas, eastern New Mexico, the panhandle of Oklahoma, western kansas, eastern Colorado, and central and western nebraska) is being studied under the Six-State High Plains-Ogallala Aquifer Regional Resources Study. Legal and institutional problems of management of the series of aquifers underlying the Rio Grande between the Elephant Butte Dam and Fort Quitman in New Mexico and Texas, and extending into Mexico are explored Initiation of Management would be difficult because of user attitudes and lack of understanding.

REFERENCE: Banks, Harvey O. (Pres., Water Resources Div., Camp Dresser and McKee, Inc., Walnut Greek, Calif.), "Management of Interstate Aquifer Systems," *Journal of the Water Resources Planning and Management Division,* ASCE, Vol. 107, No. WR2, **Proc. Paper 16575,** October, 1981, pp. 563-577

U.S. Customary-SI Conversion Factors

In accordance with the October, 1970 action of the ASCE Board of Direction, which stated that all publications of the Society should list all measurements in both U.S. Customary and SI (International System) units, the following list contains conversion factors to enable readers to compute the SI unit values of measurements. A complete guide to the SI system and its use has been published by the American Society for Testing and Materials. Copies of this publication (ASTM E-380) can be purchased from ASCE at a price of $3.00 each; orders must be prepaid.

All authors of *Journal* papers are being asked to prepare their papers in this dual-unit format. To provide preliminary assistance to authors, the following list of conversion factors and guides are recommended by the ASCE Committee on Metrication.

To convert	To	Multiply by
inches (in.)	millimeters (mm)	25.4
feet (ft)	meters (m)	0.305
yards (yd)	meters (m)	0.914
miles (miles)	kilometers (km)	1.61
square inches (sq in.)	square millimeters (mm^2)	645
square feet (sq ft)	square meters (m^2)	0.093
square yards (sq yd)	square meters (m^2)	0.836
square miles (sq miles)	square kilometers (km^2)	2.59
acres (acre)	hectares (ha)	0.405
cubic inches (cu in.)	cubic millimeters (mm^3)	16,400
cubic feet (cu ft)	cubic meters (m^3)	0.028
cubic yards (cu yd)	cubic meters (m^3)	0.765
pounds (lb) mass	kilograms (kg)	0.453
tons (ton) mass	kilograms (kg)	907
pound force (lbf)	newtons (N)	4.45
kilogram force (kgf)	newtons (N)	9.81
pounds per square foot (psf)	pascals (Pa)	47.9
pounds per square inch (psi)	kilopascals (kPa)	6.89
U.S. gallons (gal)	liters (L)	3.79
acre-feet (acre-ft)	cubic meters (m^3)	1,233

STOCHASTIC HYDROLOGIC MODEL FOR DROUGHT MANAGEMENT

By Robert M. Hirsch[1]

INTRODUCTION

Long-range water-supply forecasting is an integral part of drought management and of water supply management itself. The problems of long-range water-supply forecasting are quite different from those of short-range flow forecasting which is typically used for flood warning and hydropower scheduling. In short-range forecasting (where time horizons are a few hours to a few days), the meteorological events which are responsible for the flow of interest have already occurred or will occur within the time span for which meteorologists have substantial forecast skill. Consequently the problems are those of rapid data acquisition, and computation. The forecast output is appropriately a single trace of future flows (a hydrograph) with some error bands about it which are the result of input error, model error and the meteorological forecast uncertainty.

In contrast, the problems of long-range forecasts are those of understanding the current state of the watershed (streamflow and moisture storage in snow, soil, and aquifers) and incorporating that information with the very broad range of meteorological conditions (for which there is almost no forecasting skill) that may occur in the future. The output of a long-range forecasting model must take one of two possible forms: (1) Probability distributions for streamflow volumes during some forecast periods; or (2) a set of plausible streamflow traces covering the forecast period, each trace having an assigned probability. The presentation of a single "most likely" hydrograph is of no value in long-range forecasting. What is needed is not information on what is most likely but rather information on the likelihood (probabilities) of certain specified outcomes.

There are three broad classes of forecasting methods which are applicable to the long-range water-supply forecasting problem. The three classes are listed here in order of increasing difficulty of application. It should be noted here that little, if anything, is known about the relative appropriateness or forecasting skill of these three classes in comparison to each other.

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1. Use historic streamflow traces for the appropriate time of year as a set of possible future streamflow traces, assigning probabilities to each. These probabilities must sum to 1.0 and they may be based on information about antecedent rainfall, soil or ground-water storage associated with each historical trace. The simplest approach is to make all historic traces equiprobable. These methods were applied by Hirsch (4).

2. Develop a stochastic model of streamflow based on the historic record. Initialize the model's state variables to present conditions and use a random number generator to produce multiple (equally likely) traces from this one initial condition. The simplest model is the autoprojective model, where past streamflow alone is then used to forecast future streamflow. This is the method that will be developed in the present paper. A more complicated form is to use past streamflow as well as current or past values of other related state variables (snowpack, soil moisture, ground-water levels). Certainly where snowmelt is a major contributor to streamflow, this multivariate approach would be preferred to univariate autoprojection.

3. Develop and calibrate a deterministic hydrologic model of the river basin where the model inputs are the meteorological inputs to the basin (rain, snow, temperature, humidity, wind), and the output is streamflow. Such models have a number of state variables representing storages in the basin (snowpack, aquifers, soil zones). By simulating the antecedent hydrologic conditions (for several months), a set of starting values for the state variables are determined and then a set of historic meteorological traces (with assigned probabilities) are used as inputs to the model in order to produce a set of streamflow output traces (with the same assigned probabilities). Such a procedure has been used by the National Weather Service (6) and is known as ESP (extended streamflow prediction).

LT-ARMA (1,1) Cyclic Flow Model

The remainder of this paper deals with a stochastic flow model which was first described by Hirsch (5). It is a model of monthly streamflow volumes and the application of it described here and in the previous paper is to the Potomac River at Point of Rocks, Maryland. The probability distribution of monthly volumes is modeled as a two-parameter lognormal distribution with a different mean and variance in each month. The serial dependence is modeled in a form closely related to the autoregressive moving average (ARMA) process described by Box and Jenkins (2). The model of streamflow volumes consists of Eqs. 1 and 2:

$$Z_i = \phi Z_{i-1} + e_i - \theta_j e_{i-1} \quad \ldots \quad (1)$$

$$X_i = \exp\,[m_j + Z_i S_j] \quad \ldots \quad (2)$$

The streamflow volume in month, i, is denoted X_i in which i may take on integer values (..., −2, −1, 0, 1, 2, ...); Z_i = a transformation of X_i; m_j and S_j^2 = the mean and variance respectively, of the natural log of streamflow volume in month, j, in which j may take on integer values (1, 2, ..., 11, 12) and j is related to i by $i = 12 \cdot g + j$ in which g = an integer. The variables, e_i, are independent and normally distributed with mean, 0, and variance β_j^2.

The Z_i are normally distributed with mean 0, variance 1, but are serially correlated. The total number of independent parameters is 37 (ϕ, 12 $\theta_j's$, 12 $m_j's$, and 12 $S_j's$, the 12 $\beta_j's$ are determined by ϕ and the 12 $\theta_j's$). The method of parameter estimation used for this model is given by Hirsch (5) under the heading of the "LT-ARMA" model. The important point to be noted about the estimates of the parameters of Eq. 1 is that the lag one serial correlation between the log transformed flow values for all 12 months are exactly preserved in the long run, and that serial correlation of log flow values at lags 2–12 are preserved in a least-squares sense. The serial correlation structure of Z_i may be derived from (1). Let $\rho_j(l)$ be the correlation between Z_i and Z_{i-l}, for $i = 12 \cdot g + j$:

$$\rho_j(l) = E[Z_i Z_{i-l}] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

then $\rho_j(l) = \phi^{l-1}\ [\phi - \theta_{j+1-l}\ \beta^2_{j-l}]$ $l = 1, 2, \ldots$; and $j = 1, 2, \ldots, 11, 12$. It should be noted that where the index of any model parameter θ, β, m or S is outside the range (1, 2, ..., 11, 12), it is assumed to be recomputed by adding or subtracting integer multiples of 12 to bring it into the range (1, 2, ..., 11, 12). For example, $\rho_2(3) = \phi^2\,[\phi - \theta_{12}\beta^2_{11}]$.

Using related notation, the stationary ARMA (1,1) model is expressed

$$Z_i = \phi Z_{i-1} + e_i - \theta e_{i-1}$$

and $\rho(l) = \phi^{l-1}\ [\phi - \theta\beta^2]$ in which $\rho(l)$ = the lag l correlation coefficient; and β^2 = the variance of the e_i values.

Use of ARMA (1,1) Cyclic Model in Forecasting

The derivation of the forecasting method will begin with the problem of forecasting Z values and then proceed to the forecasting of X (streamflow volume). Given the model for Z (Eq. 1), assume that Z_{i-1} and e_{i-1} are known, then

$$E[Z_i | Z_{i-1}, e_{i-1}] = \phi Z_{i-1} - \theta_j e_{i-1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4a)$$

$$\text{Var}[Z_i | Z_{i-1}, e_{i-1}] = \beta_j^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4b)$$

in which $E[Z_i|Z_{i-1}, e_{i-1}]$ represents the expectation of Z_i conditional on Z_{i-1} and e_{i-1} and $\text{Var}[Z_i|Z_{i-1}, e_{i-1}]$ is the conditional variance of Z_i given Z_{i-1} and e_{i-1}.

Define $\hat{Z}_i \equiv E[Z_i|Z_{i-1}, e_{i-1}]$, then $\hat{Z}_i = \phi Z_{i-1} - \theta_j e_{i-1}$ . . . . . (5)

The one-step ahead forecast error is $Z_i - \hat{Z}_i$. Subtracting Eq. 5 from Eq. 1 we have

$$Z_i - \hat{Z}_i = e_i \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

So e_i = the one-step ahead forecast error. Therefore, given the model, Z_{i-1} and e_{i-1}, are the necessary and sufficient information for making the forecast, $\hat{Z}_i$, and this forecast and the realization, Z_i, are necessary and sufficient information for determining e_i. It would appear that there is some difficulty in making the first forecast because only Z_1 would be known. In practice, an arbitrary choice of e_1 values can be made for the initial month of a historical time series and the computation of successive forecasts, $\hat{Z}_i$, and forecast errors,

e_i, can be carried out up to the present month. For any reasonable set of parameter values the choice of e_1 will make an insignificant contribution to the e_i value several months later.

Thus, by making predictions and computing errors in an interative fashion through the entire history of the time series $Z_1, \ldots Z_{i-2}, Z_{i-1}$ up to time $i - 1$, it is possible to "know" the values of e_{i-1} even though it is not a variable which is subject to direct measurement.

The conditional expectations and variances for forecasts more than one month ahead are

$$E[Z_{i+k}|Z_{i-1}, e_{i-1}] = \phi^k [\phi Z_{i-1} - \theta_j e_{i-1}] \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

$$\text{Var}[Z_{i+k}|Z_{i-1}, e_{i-1}] = \beta_{j+k}^2 + \sum_{h=1}^{k} \beta_{j+k-h}^2 (\phi^h - \phi^{h-1}\theta_{j+k-h+1})^2 \quad \ldots\ldots (8)$$

for $k = 1, 2, 3, \ldots$.

Recalling that the purpose of the forecasting exercise is not to produce a most likely streamflow trace but rather to produce a risk assessment, then these forecasts must be translated into cumulative distribution functions (CDF). It can be shown that the conditional distribution of Z_i (given Z_{i-1}, e_{i-1}) is approximately normal with mean $\hat{Z}_i$ and variance β_j^2. Thus, knowing the relationship between X_i and Z_i (Eq. 2), one can specify completely the CDF for X_i. Let the unconditional CDF of X_i be denoted $F_{X_i}(x) \equiv \text{Prob}(X_i \leq x)$, and the conditional CDF of X_i be denoted $F_{X_i}(x|Z_{i-1}, e_{i-1}) \equiv \text{Prob}(X_i \leq x|Z_{i-1}, e_{i-1})$. Let the standard normal CDF be denoted $F_z(t)$. $F_z(t) = \int_{-\infty}^{t} 1/\sqrt{2\pi}\ e(-u^2/2)du$.

$$\text{Then} \quad F_{X_i}(x) = F_z\left(\frac{\ln x - m_j}{S_j}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

$$\text{and} \quad F_{X_i}(x|Z_{i-1}, e_{i-1}) = F_z\left(\frac{\dfrac{\ln x - m_j}{S_j} - \hat{Z}_i}{\beta_j}\right) \quad \ldots\ldots\ldots\ldots (10)$$

and more generally the conditional CDF for any number of time steps ahead is

$$F_{X_{i+k}}(x|Z_{i-1}, e_{i-1}) = F_z\left(\frac{\dfrac{\ln x - m_{j+k}}{S_{j+k}} - E[Z_{i+k}|Z_{i-1}, e_{i-1}]}{(\text{Var}[Z_{i+k}|Z_{i-1}, e_{i-1}])^{1/2}}\right) \quad \ldots\ldots (11)$$

The forecast of flows (x_i as opposed to Z_i) can be expressed in either of two ways. One is to give a median forecast $\hat{x}_i(\text{med})$ in which

$$\hat{x}_i(\text{med}) \equiv \exp(m_j + S_j \hat{Z}_i) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

This forecast is the median of the conditional distribution, that is

$$\text{Prob}[X_i \leq \hat{x}_i(\text{med})] = 0.50 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

The other is the expected value or mean forecast $\hat{x}_i(\text{mean})$ in which

$$\hat{x}_i(\text{mean}) = \exp\left[\frac{1}{2}\cdot S_j^2\beta_j^2 + m_j + S_j\hat{Z}_i\right] \quad (14)$$

This forecast is the mean of the conditional distribution.

The conditional standard error of these $\hat{x}_i$(mean) forecasts is

$$\text{SE}(\hat{x}_i) = \exp\left[m_j + S_j\hat{Z}_i\right]\sqrt{\lambda^2(\lambda^2+1)} \quad (15)$$

in which $\lambda^2 = \exp[S_j^2\beta_j^2] - 1$.

Note that λ is the conditional coefficient of variation of x_i and it is not dependent on the streamflow forecast itself. The conditional coefficient of skewness of x_i is γ_x in which $\gamma_x = \lambda^3 + 3\lambda$; thus, the skew does not depend on the flow forecast.

The other mode of application of the model is to simulate multiple equally likely streamflow traces. For simplicity of computation rewrite the generating Eqs. 1 & 2 as

$$Z_{i+k-1} = \phi Z_{i+k-2} + \beta_{j+k-1}\epsilon_{i+k-1} - \theta_{j+k-1}\beta_{j+k-2}\epsilon_{i+k-2} \quad (16)$$

$$X_{i+k-1} = \exp\left[m_{j+k-1} + Z_{i+k-1}S_{j+k-1}\right]; \quad k = 1, 2, \ldots, n \quad (17)$$

in which $Z_{i-1} = \ln x_{i-1} - m_{j-1}/S_{j-1}$; $\epsilon_{i-1} = e_{i-1}/\beta_{j-1}$; and ϵ_i, ϵ_{i+1}, ϵ_{i+2}, ..., ϵ_{i+n-1} are realization of white noise (standard normal independent random variables). By generating N such sequences from a random number generator, one may produce N equally likely sequence of n flow values X_i, X_{i+1}, ..., X_{i+n-1}.

Each of these N sequences can then be used as inputs to a simulation of the water-supply storage and delivery system. By doing continuity accounting for each component of the system given present storages and projected withdrawal rates, one may determine m, the number of simulations that have some particular outcome (say zero storage). The estimated probability of that outcome becomes m/N.

Examples

The model parameters were fit to the 81-yr monthly flow record for the Potomac River at Point of Rocks, Maryland. The full set of parameters and the sample values of $\rho_j(1)$ are given in Table 1. The 12 values of $\rho_j(1)$ are shown to demonstrate their wide month-to-month differences. In Monte Carlo experiments it was determined that such a wide variation is very unlikely to have occurred by chance if all 12 values $\rho_j(1)$ were equal. Noting the variation in β_j^2 values it is clear that the predictive ability of the model for month ahead forecasts is highly dependent on the month. For example, forecast variance for April is 0.91 but for November it is 0.56.

The explanatory power of this model is compared to several others in Table 2. These models all assume the same unconditional flow distribution (lognormal) with the same m_j and S_j parameters given in Table 1. The measure of the explanatory power is in all cases the following estimate of the error variance

$$1/960\sum_{i=13}^{972}\hat{e}_i^2$$

in which $\hat{\epsilon}_i$ = the estimated error term for month i. The reason that the first 12 values of $\hat{\epsilon}_i$ are not used is because of the arbitrary start-up rule (which was $\hat{\epsilon}_1 \equiv Z_1$). More complicated techniques involving hindcasting could have been used to prevent this "waste" of 12 values but given the wealth of data it was felt to be unnecessary. The following six models are defined:

ARMA (1,0) stationary $Z_i = \phi Z_{i-1} + \beta\epsilon_i$
ARMA (1,0) cyclic $Z_i = \phi_j Z_{i-1} + \beta_j \epsilon_i$
ARMA (1,1) stationary $Z_i = \phi Z_{i-1} + \beta\epsilon_i - \theta\beta\epsilon_{i-1}$
ARMA (1,1) cyclic $Z_i = \phi Z_{i-1} + \beta_j \epsilon_i - \theta_j \beta_{j-1} \epsilon_{i-1}$
ARMA (2,1) stationary $Z_i = \phi_1 Z_{i-1} + \phi_2 Z_{i-2} + \beta\epsilon_i - \theta\beta\epsilon_{i-1}$
ARMA (2,2) stationary $Z_i = \phi_1 Z_{i-1} + \phi_2 Z_{i-2} + \beta\epsilon_i - \theta_1\beta\epsilon_{i-1} - \theta_2\beta\epsilon_{i-2}$

TABLE 1.—Statistics and Model Parameters for ARMA (1,1) Cyclic Model of Potomac River at Point of Rocks, Maryland—Monthly Volumes

Month (1)	Index j (2)	Mean of log volumes m_j (3)	Standard deviation of log volumes S_j (4)	θ_j (5)	β_j (6)	Lag one correlation coefficient $\rho_j(1)$ (7)
October	1	5.50	0.838	0.499	0.905	0.357
November	2	5.63	0.746	0.040	0.745	0.667
December	3	6.15	0.776	0.085	0.756	0.653
January	4	6.57	0.633	0.067	0.749	0.661
February	5	6.74	0.602	0.734	0.886	0.288
March	6	7.16	0.483	0.584	0.940	0.242
April	7	6.95	0.511	0.505	0.954	0.254
May	8	6.64	0.576	0.455	0.949	0.286
June	9	6.20	0.614	0.484	0.954	0.264
July	10	5.67	0.558	0.128	0.811	0.589
August	11	5.54	0.672	0.423	0.884	0.421
September	12	5.29	0.672	0.187	0.829	0.554

TABLE 2.—Results of Fitting Six Alternative Stochastic Models to Potomac River Data. Numbers in Parentheses Indicate Ranks (1—best, 6—worst)

Model (1)	Error variance (2)	Number of ARMA parameters (3)	AIC (4)
ARMA (1,0) stationary	0.810 (6)	1	2,523.94 (6)
ARMA (1,0) cyclic	0.782 (2)	12	2,512.29 (5)
ARMA (1,1) stationary	0.792 (5)	2	2,504.01 (2)
ARMA (1,1) cyclic	0.752 (1)	13	2,476.61 (1)
ARMA (2,1) stationary	0.791 (4)	3	2,505.77 (3)
ARMA (2,2) stationary	0.791 (3)	4	2,507.28 (4)

in which ϵ_i = in all cases white noise. All of the stationary ARMA models were fit using maximum likelihood estimation, the ARMA (1,0) cyclic model was fit to assure long-run preservation of the sample values of $\rho_j(1)$ for all 12 months, and the ARMA (1,1) cyclic model was fit by the procedures previously mentioned.

One may argue that the relative size of the forecast variances are simply a consequence of the number of parameters used and, in fact, for these six models there is a perfect inversion of ranks between forecasts variance and number of ARMA parameters (lower variance corresponding to higher number of parameters). Hipel (3) suggests using the Akaike Information Criterion (AIC) (1) in model selection to incorporate the criterion of parsimony along with that of error minimization. They recommend selecting the model that gives the minimum value of the AIC defined by: AIC = -2 ln (maximum likelihood) + $2K$, in which K = the number of ARMA parameters. Using this criterion, rather than minimum variance alone, the ARMA (1,1) cyclic remains the preferred model, the rather parsimonious ARMA (1,1) stationary model moves from being the fifth best model to the second best. Undoubtedly, with a substantially shorter data set, the ARMA (1,1) stationary model may be preferred. The appropriate degree of model complexity remains a major problem in the construction of forecasting models.

Applications

Three different conditional CDF's (for three different antecedent conditions) are shown in Figs. 1, 2, and 3. All are historical cases in which the forecast is of the July volume based on volumes for the entire record up to and including June. Also shown on the figures is the unconditional July CDF. Table 3 gives a description of the three situations. In the first case there was a sustained period of very high flow through June—so the July forecast is quite high. The

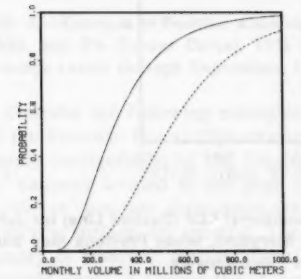

FIG. 1.—Unconditional CDF (Solid Line) and Conditional CDF (Dashed Line) for July Volumes for Potomac River at Point of Rocks, Maryland, when Previous May and June Volumes were 2,042 $\times$ 10^6 m^3 and 1,408 $\times$ 10^6 m^3, Respectively

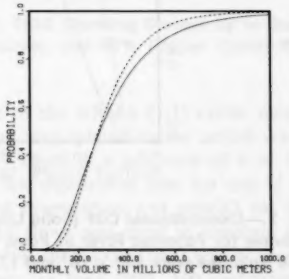

FIG. 2.—Unconditional CDF (Solid Line) and Conditional CDF (Dashed Line) for July Volumes for Potomac River at Point of Rocks, Maryland, when Previous May and June Volumes were 341 $\times$ 10^6 m^3 and 512 $\times$ 10^6 m^3, Respectively

conditional probability of a volume below 200 m^3 is only 0.017 as compared to the unconditional probability of 0.251.

In the second case the June volume was almost exactly the historic median value (Z_{i-1} is very near 0.0), however, the previous month's volume was quite low. If the forecasting model were ARMA (1,0) rather than ARMA (1,1), the

TABLE 3.—Examples of Three Forecasts for Potomac July Streamflow Volumes in 10^6 m^3

Z_{i-2} (1)	Z_{i-1} (2)	Z_i (3)	x_{i-2} (4)	x_{i-1} (5)	$\hat{x}_i$ (median) forecast (6)	$\hat{x}_i$ (mean) forecast (7)	Standard error of $\hat{x}_i$ (mean) (8)	Probability ($X_i < 200$) (9)
1.696	1.705	1.047	2,042	1,408	522	578	275	0.017
−1.414	0.060	−0.032	341	512	286	316	151	0.215
−0.614	−1.715	−0.993	540	172	167	185	88	0.654
Unconditional					291	340	205	0.251

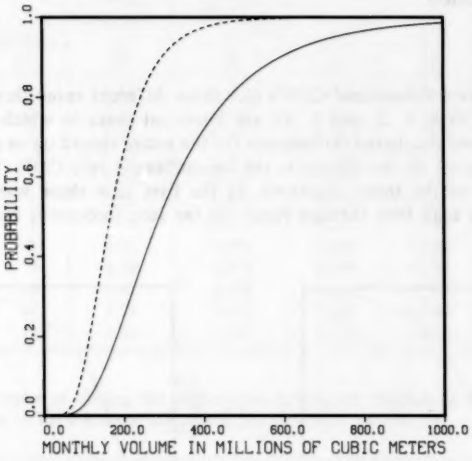

FIG. 3.—Unconditional CDF (Solid Line) and Conditional CDF (Dashed Line) for July Volumes for Potomac River at Point of Rocks, Maryland, when Previous May and June Volumes were 540 × 10^6 and 172 × 10^6 m^3, Respectively

forecast value $\hat{Z}_i$ would have been positive because Z_{i-1} was positive. The ARMA (1,1) formulation contains "memory" of the very low May volume and this more than compensates for the very slightly positive June value resulting in a small negative $\hat{Z}_i$ value. In the third case the two previous months had low volumes so the forecast is for a low volume.

The conditional standard error of the log volumes for July is 0.452 ($= S_j\beta_j$), independent of past flows, and the unconditional standard error for July is 0.558 ($= S_j$). Thus in log space the knowledge of past flows always reduces forecast error. In contrast, the conditional standard error of the flows themselves depends on the history of flows. When $\hat{Z}_i$ is high the standard error is higher than the unconditional standard error, but when $\hat{Z}_i$ is near zero or negative, the standard error may be a good deal below the unconditional.

If a water-supply system had storage only sufficient for regulating the delivery of water over periods of about a month, then these CDF's would be highly useful for making risk assessments. If the storage were only sufficient for a few days, then one would need a stochastic model with a shorter time step in order to evaluate the risk of shortage. If the storage capacity were sufficient to carry over a few months, then it would be appropriate to use this kind of forecasting model in a synthetic streamflow generating mode.

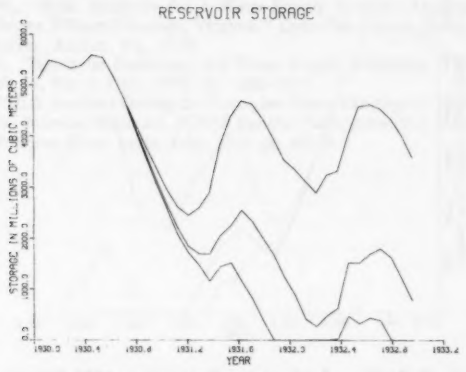

FIG. 4.—Example of Position Analysis for June, 1930, Showing Storage up to June, 1930, and 2% (Lower Curve), 10% (Middle Curve), and 50% (Upper Curve) Risk Storage Levels through September, 1932

Consider the following examples of a use of the ARMA (1,1) cyclic model of the Potomac flows. (The example is in no way related to the actual water supply configuration of the Potomac River.) There is a reservoir of 6×10^9 m^3 capacity located at the gage site, and the withdrawal rate for use is 20×10^6 m^3 per day throughout the year and evaporation and rainfall on the reservoir surface are ignored. The history of flows was used to simulate the history of reservoir storage from October 1896–June 1930 (1930 was a severe drought year). Knowing the Z_i and e_i values for June 1930, 100 realizations of streamflows through the remainder of water years 1930, 1931, and 1932 were generated using Eqs. 16 and 17. Using the end of June 1930 storage conditions for each realization, the continuity equation was used to develop 100 realizations of reservoir storages through the end of water year 1932. The 100 realizations of storage in any month may be viewed as a sample CDF. From these synthetic traces one may estimate the risk of storage falling below some specified level

in one particular month or over all months. For graphical purposes a few selected risk levels are traced out over the simulation period (Fig. 4). Given 100 realizations the 2% risk curve follows the second smallest storage values through all the months, the 10% follows the tenth smallest, and 50% follows the median. Note that any one risk level curve will not be identical with any one of the 100 traces but, rather, will jump from one to the other.

The results suggest that the probability of storage reaching zero is less then 2% through August of 1931 but it is never as great as 10% during the simulation period. Another simulation was run taking the starting time as September 1930 after three more months of continuing drought conditions (Fig. 5). The time for which the probability of zero storage exceeds 2% remains the same but the new estimate of zero storage probability now exceeds 10% for the period October–December of water year 1932.

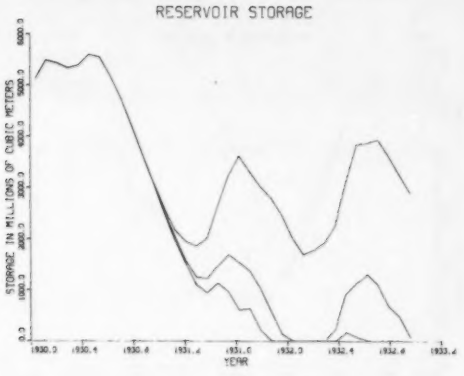

FIG. 5.—Example of Position Analysis for September, 1930 Showing Storage up to September, 1930, and 2% (Lower Curve), 10% (Middle Curve), and 50% (Upper Curve) Risk Storage Levels through September, 1932

Using such an analysis one can obtain, every month, an updated estimate of the conditional probability of storage falling below some level (such as zero) over some specified time horizon. In fact, given a large enough number of synthetic traces and a long enough time horizon, one can obtain results showing the gradual evolution from the conditional CDF of storage to the unconditional CDF of storage as the memory of the initial values of storage and flow die away.

Conclusions

With any of these tools for long-range water supply forecasting the ultimate aim is to provide information to improve the management of a water supply system. The power of the tools comes when they are used to provide information on the outcome of management options. For example, Figs. 4 or 5 could be re-created under a different withdrawal rate assumption. This kind of approach

allows for the quantification of the risk side of the risk-cost tradeoff.

There is need for more investigation into the problems of parameter estimation for stochastic forecasting models. Finding the proper balance between preservation of detailed seasonally varying phenomena (flow correlation and CDF's) with the need for parsimony and the desire to account explicitly for the uncertainty in some 36 or more parameters remains a challenging task.

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WETLANDS' USE FOR WATER MANAGEMENT IN FLORIDA[a]

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INTRODUCTION

The 2,300 sq mile (5,960 km^2) Kissimmee River Basin is located in the central portion of peninsular Florida. The river originates near Orlando and passes through a series of shallow lakes in the upper basin before emerging south of Lake Kissimmee as a gently sloping river meandering across a relatively narrow marsh flood plain south to Lake Okeechobee (as shown in Figs. 1 and 2). Numerous sloughs and small lakes drain the wet prairies adjacent to the flood plain.

In October, 1956 the Corps of Engineers (COE) released a report citing the need for flood control and water conservation in the basin (4). Due to prolonged seasonal rainfalls, inadequate secondary drainage canals, and limited outlet capacity, large areas of the watershed were periodically flooded. Tropical hurricanes, which usually occur during the rainy season, also intensified the problems.

Extensive and costly flooding occurred numerous times before the publication of the Corps of Engineers Report, e.g., in 1945, 1947, 1948, 1951, and 1953, and the expanding agricultural economy in central Florida indicated that the flood damages would only increase in the future. As part of the Corps of Engineers, Central and Southern Florida Flood Control Project, a 53-mile-long (85-km) channel, designated C-38, displaced the 96-mile (155-km) serpentine river channel, primarily to aid in regulation of the lake stages in the upper basin. The channel, completed in 1971, is 30 ft (9.2 m) deep, averages 200 ft (61 m) in width, and steps down in six pools by six primary control structures and tie-back levees. Prior to channelization, about 130,000 acres (5.3×10^8 m^2), 30% of the lower basin, were wetlands, and at peak stages the flood plain approached

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2 miles (3.2 km) in width, supporting the extensive marsh system. By 1972, only 80,000 acres (3.2×10^8 m^2), 18% of the basin, remained in the wetlands, although much of the drained land, about 80%, is upland and could have been drained without the construction of C-38 (6). The combined upland drainage and flood control system has changed the basin hydrology from one of upland/flood plain retention and slow runoff to one characterized by upland/flood plain drainage and rapid runoff (3). Since the river contributes the major portion of surface inflow and a substantial portion of nutrients to lake Okeechobee, it is difficult to overstate the significance of the Kissimmee River to the well-being of Lake Okeechobee.

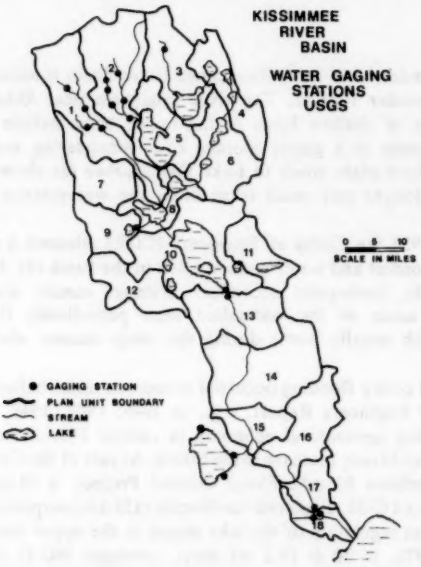

FIG. 1.—Upper and Lower Basins Showing Soil Conservation Service Planning Units

Lake Okeechobee, with over 700 sq miles (1,800 km^2) of surface area, functions to integrate the unique hydrologic system of the south Florida region by providing a principal freshwater storage site. Lake water is used directly by several cities for a domestic water supply, for supplying agricultural irrigation water, for supplying the Everglades National Park with at least 315,000 acre-ft (3.88×10^8 m^3) per year, and for recharging in dry years the Biscayne aquifer, which in turn supplies freshwater to 2,200,000 residents of Broward and Dade Counties. In addition, Lake Okeechobee provides recreation, commercial, and sport fishing, and supports many and varied forms of fish and wildlife (including several endangered species) (3). Finally, it serves as a major flood control basin for the area. Thus, the quality and quantity of water in Lake Okeechobee is an

extremely important issue for all south Florida.

As a result of degrading water quality in C-38 and reports that Lake Okeechobee is in an early eutrophic state, The Special Project to Prevent Eutrophication

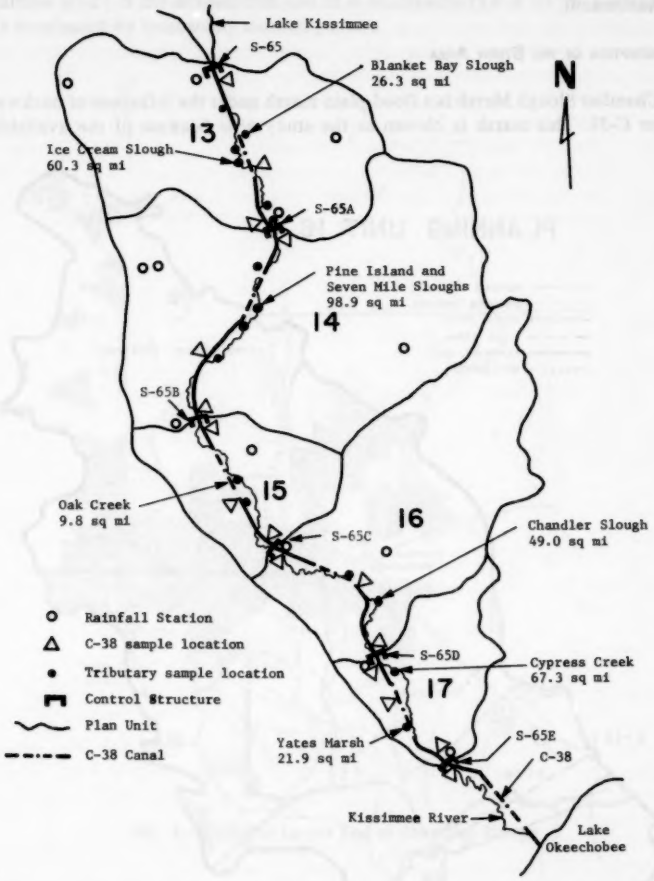

FIG. 2.—Lower Basin, Including Sampling Sites Used by South Florida Water Management District

of Lake Okeechobee was funded by the 1973 Florida Legislature with the final report published in November, 1976 (3). A general recommendation of this report is: "A surface water management scheme based on detention of runoff, restrictions on surface water discharge rates, and routing of as much flow as possible

through natural or manmade marshes should be implemented basinwide."

This paper evaluates the ability of these marshes to serve as quantity and quality control areas in the lower Kissimmee Basin and addresses specific questions concerning trade-offs among drainage, flood control, and water quality enhancement.

Description of the Study Area

Chandler Slough Marsh is a flood-plain marsh under the influence of backwater from C-38. This marsh is chosen as the study area because of the availability

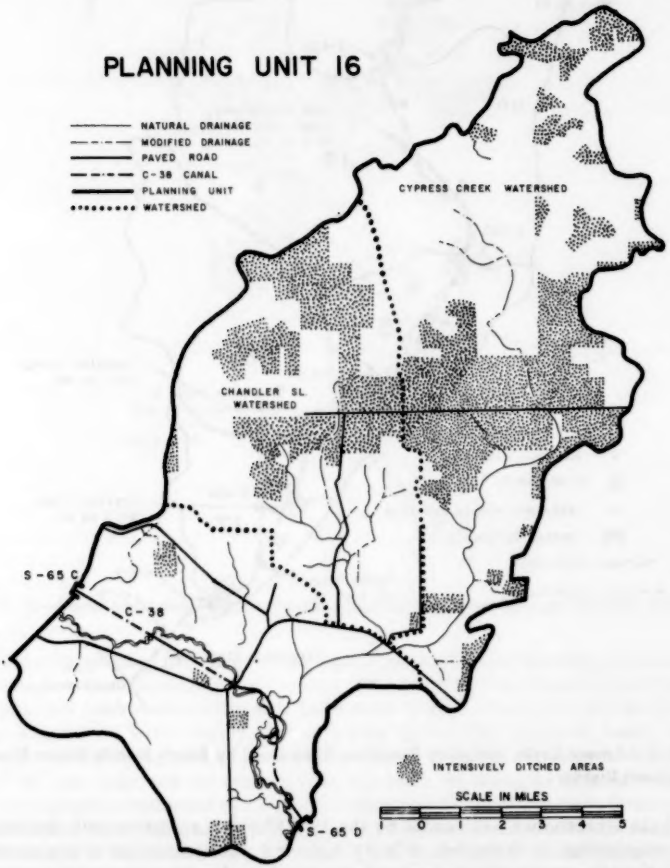

FIG. 3.—Location of Chandler Slough Within SCS Planning Unit 16

of inflow and water quality data from the South Florida Water Management District (SFWMD). The marsh, shown in Figs. 3 and 4, is located in SCS Planning Unit 16 and is a portion of Chandler Slough between C-38 and United States Highway 98. The 1,000-acre (4.05×10^6 m^2) marsh [at a stage of 29.5 ft (9.0 m) above MSL] is the downstream end of a 74,000-acre (3.0×10^8 m^2) drainage area dominated by intensively ditched pasture.

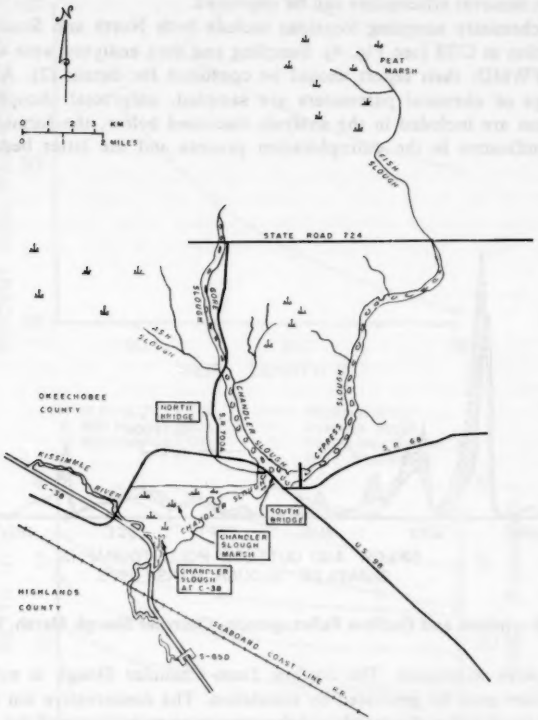

FIG. 4.—Detail of Lower End of Chandler Slough

For the following analysis, the storage/treatment portion of the Storm Water Management Model (SWMM) (6) is used to simulate the hydrology and water quality of Chandler Slough Marsh. SWMM is used because the marsh can be modeled as a natural (irregular) storage basin with plugflow, first-order nutrient removal, and surface evaporation. The depth-area relationship in the model is a series of 11 depths with corresponding areas which are derived with transects of Chandler Slough Marsh supplied by the SFWMD. The depth-discharge relationship is represented by a power function.

Water Quality Control

The evaluation of the marsh's ability to serve as a water quality control unit is in two phases: (1) Mass loadings to and from Chandler Slough Marsh are calculated to determine the nutrient budget and thus the effectiveness of the marsh in nutrient removal in its present state; and (2) a first-order uptake coefficient is estimated, and the outlet control structure is modeled to determine if nutrient removal efficiencies can be improved.

Water chemistry sampling locations include both North and South Bridges and a station at C-38 (see Fig. 4). Sampling and data analyses were conducted by the SFWMD; their report should be consulted for details (2). Although a wide range of chemical parameters are sampled, only total phosphorus and chloride ion are included in the analysis discussed below, the former because of its significance in the eutrophication process and the latter because it is

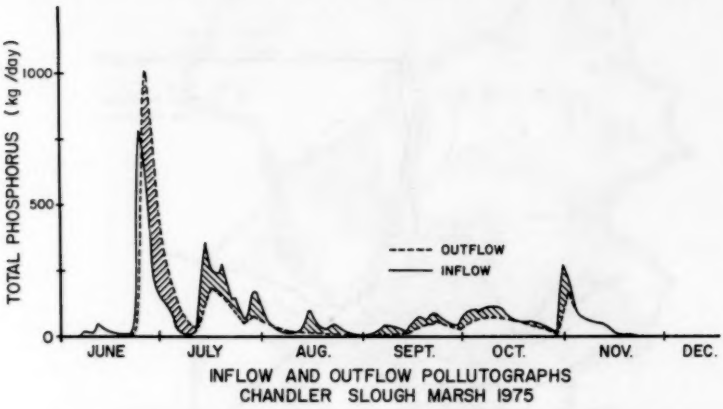

FIG. 5.—Inflow and Outflow Pollutographs, Chandler Slough Marsh, 1975

a conservative substance. The outflow from Chandler Slough is not gauged and therefore must be generated by simulation. The conservative ion is diluted or concentrated within the marsh and thus serves as an indicator of the accuracy of the outflow predictions.

Data are intermittent; therefore, concentrations for days without data are generated by linear interpolation. By knowing the concentration of the parameters and the measured inflows and predicted outflows, mass loadings to and from Chandler Slough Marsh are found for 1975 and 1976.

Net total phosphorus removals for 1975 and 1976 are 6.7% and 34.8%, respectively. This is equivalent to an "uptake" by the marsh for 1975 and 1976 of 5.1 lb/acre (5.7×10^{-4} kg/m^2) and 17.2 lb/acre (1.9×10^{-4} kg/m^2), respectively. Total imports of total phosphorus are 16.2 tons (1.5×10^4 kg) and 24.7 tons (2.2×10^4 kg) while exports are 15.1 tons (1.4×10^4 kg) and 16.1 tons (1.5×10^4 kg) for 1975 and 1976, respectively. The wet-season first-order

decay coefficient at 20° C for total phosphorus is found to be on the order of 0.2 day^{-1} by varying the decay coefficient and comparing SWMM predicted mass export with the mass export found above using actual concentration data. Pollutographs of inflow and outflow, for June through December, 1975, are shown in Fig. 5. This figure shows the "first flush" effect at the beginning

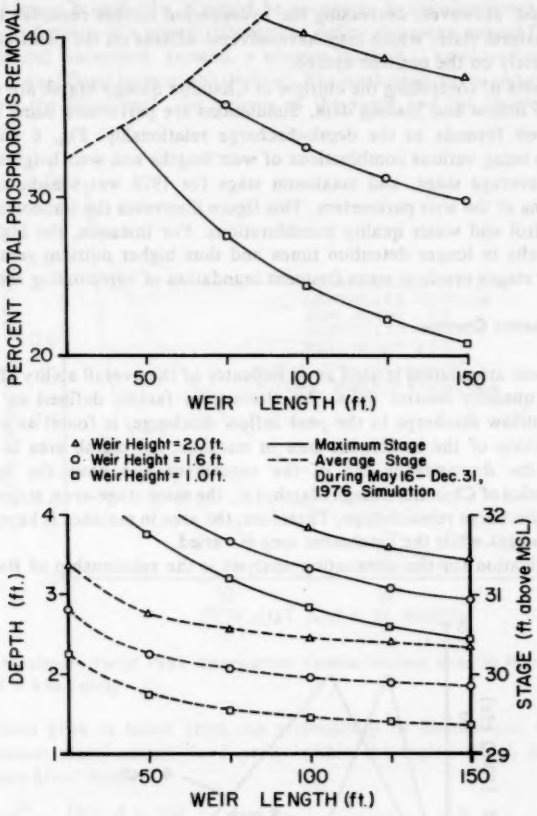

FIG. 6.—Percent Total Phosphorus Removal, Average Stage and Maximum Stage, Chandler Slough Marsh, 1976 Wet Season (Simulated)

at the wet season, where undoubtedly much of the previous year's deposition of total phosphorus is released.

Apparent chloride ion removals of 5.0% and 8.6% are observed for 1975 and 1976. These apparent removals suggest dilution by unaccounted-for inflows and thus even a lower actual removal of total phosphorus than indicated.

The marsh in its present state may periodically release a large portion of accumulated phosphorus, as is the case at the beginning of the 1975 wet season. "Flushing" of the marsh is speculated to occur due to large inflows after a period of low water (i.e., large portions or all of the marsh area are dry). If this is the case, stage regulation with an outlet structure could minimize dry out and possibly reduce the release of phosphorus by decreasing the hydroperiod. However, decreasing the hydroperiod further removes the marsh from its natural state, which may have adverse effects on the plant community and ultimately on the nutrient uptake.

The effects of controlling the outflow of Chandler Slough Marsh are evaluated using 1976 inflow and loading data. Simulations are performed using the broad crested weir formula as the depth-discharge relationship. Fig. 6 summarizes the results using various combinations of weir lengths and weir heights. Percent removal, average stage, and maximum stage for 1976 wet season are shown as functions of the weir parameters. This figure illustrates the tradeoffs between flood control and water quality considerations. For instance, the highest weir height results in longer detention times and thus higher nutrient removal, but the higher stages result in more frequent inundation of surrounding areas.

Water Quantity Control

Flood peak attenuation is used as an indicator of the overall ability of marshes to act as quantity control units. An attenuation factor, defined as the ratio of peak outflow discharge to the peak inflow discharge, is found as a function of the fraction of the catchment area in marshes. All marsh area is assumed to be at the downstream end of the catchment and have the hydrologic characteristics of Chandler Slough Marsh, i.e., the same stage-area, stage-volume, and stage-discharge relationships. Therefore, the area in marshes is kept approximately constant while the catchment area is varied.

The motivation for the attenuation analysis is the relationship of Barnes and

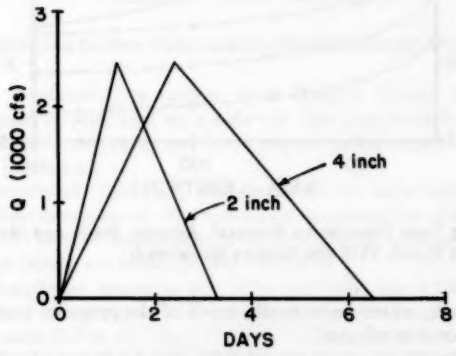

FIG. 7.—Flood Hydrographs for a 75 Sq Mile (194 kg^2)-Catchment with Runoff Equal to 2 in. (150.8 mm) and 4 in. (101.6 mm) (Peak given by Eq. 1)

Golden (1) for attenuation of the mean annual flood. They derive a curve for the reduction of the mean annual flood discharge in relation to the percent of the drainage area in lakes and swamps by comparing discharge records for drainage basins containing lakes and swamps (swamp is presumed to also mean marsh) with otherwise equivalent basins.

Since the entire lower Kissimmee Basin has discharge records only at the upper and lower boundaries, it would be necessary to simulate, on a long-term basis, the hydrology of a lateral tributary to define the mean annual flood event for that lateral watershed. Instead, a simpler method is employed to estimate the mean annual flood hydrograph (inflow). The method involves certain assumptions about the flood peak, the volume of the event and the geometry of the hydrograph.

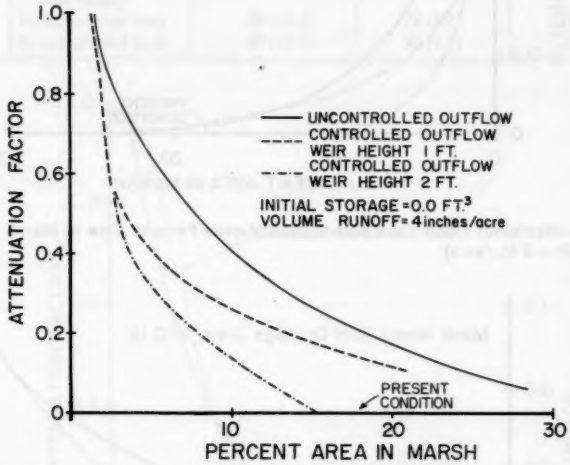

FIG. 8.—Maximum Flood Peak Attenuation Versus Percent Area in Marsh (Volume of Runoff = 4 in./acre)

The flood peak is taken from the relationship of Barnes and Golden for the maximum mean annual flood peaks within the region which includes the Kissimmee River Basin:

$$Q = 285\,A^{0.5}, \quad 10 \le A \le 100 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which Q = mean annual flood peak, cfs; and A = catchment area, in sq miles.

Data concerning the return periods of total flood volumes are not available; therefore, volume assumptions must be made. Rainfall of 5 in. (127 mm) is assumed to be a reasonable estimate of the precipitation which might produce the mean annual flood. The 5-in. amount is the maximum 24-h rainfall to be expected once in 2 yr (5). The return period of the mean annual flood is 2.33 yr. It is important to remember that the 2-yr maximum 24-hr rainfall may not

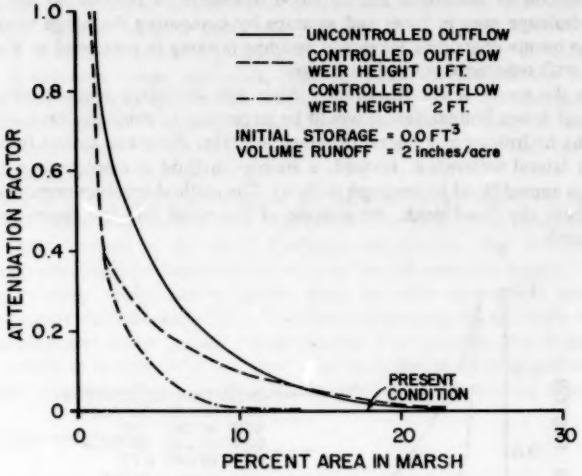

FIG. 9.—Maximum Flood Peak Attenuation Versus Percent Area in Marsh (Volume of Runoff = 2 in./acre)

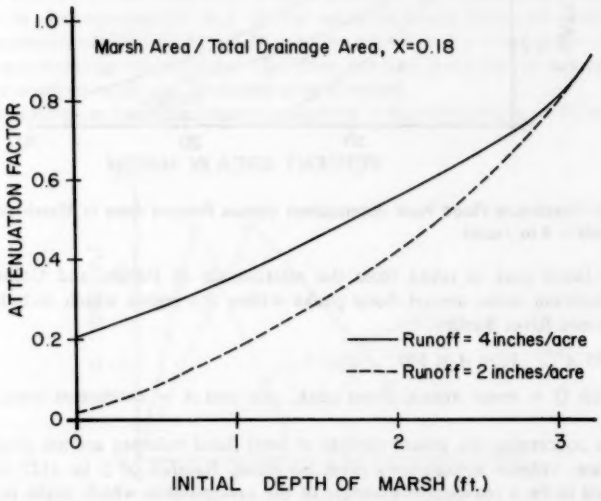

FIG. 10.—Attenuation Factor Versus Initial Depth of Marsh

be related to a flood with a similar return period, since the magnitude of a flood is also controlled by other factors such as storm duration and antecedent conditions. The ratio of runoff to rainfall for wet years (rainfall greater than average) from a nearby watershed is approximately 0.4. Applying this ratio

TABLE 1.—Outlet Relationship Modeled in Study

Case (1)	Outflow (2)	Weir length, in feet (meters) (3)	Weir height, in feet (meters) (MSL) (4)	Weir coefficient (5)
1	uncontrolled existing relationship	215 (65.6)	29 (8.8)	2.1
2	broad crested weir	40 (12.2)	29 (8.8)	1.5
3	broad crested weir	40 (12.2)	30 (9.2)	1.5

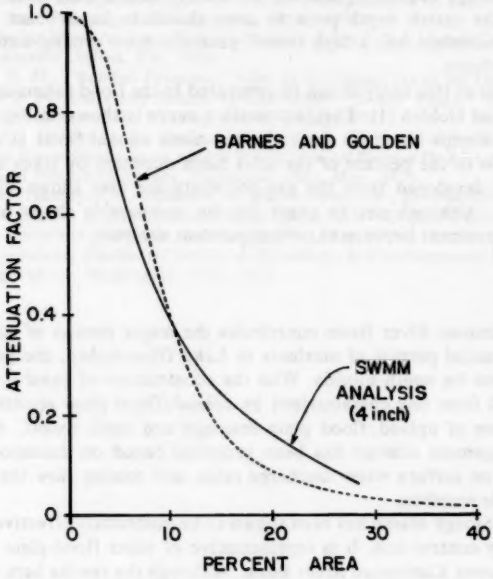

FIG. 11.—SWMM Analysis (Percent Area in Marsh) Compared with Barnes and Golden Analysis (Percent Area in Lakes and Swamps) for Attenuation of Mean Annual Flood

to the 5 in. (12.7 cm) yields runoff of 2 in. (7.1 cm). Using the antecedent conditions with the highest runoff potential, the SCS Runoff Curve Number Method (8) gives runoff equal to 3.9 in. (9.9 cm). Hence, the total volume of runoff is estimated to be between 2 in. (5.1 cm) and 4 in. (10.2 cm). These

two runoff values are used as possible volumes of the flood.

The shape of the flood hydrograph is derived from a dimensionless triangular unit hydrograph where 37.5% of the runoff volume is under the rising limb of the unit hydrograph (8). The two flood hydrographs for a 75 sq mile (190 km^2) catchment are shown in Fig. 7.

Three outlet relationships are modeled using the weir equation as described in Table 1. Maximum attenuation, as it relates to the percent of the catchment in marshes, is shown in Figs. 8 and 9 for the two runoff volume assumptions. Maximum attenuation occurs when the initial marsh storage is zero. The arrow represents the present condition of SCS Planning Unit 16 with 18% of the catchment in marshes. It is evident that the modeled control structures increase the single event attenuation achieved by the marsh, although they rapidly lose their effectiveness when the percent of catchment in marshes decreases below about 5%.

The effect of the initial condition of the marsh on its ability to reduce the flood peak for the present condition is shown in Fig. 10. As initial depth increases, available storage decreases, and the attenuation factor increases. It is realistic to expect the marsh depth prior to most floods to be at least 1 ft (0.3 m) since the watershed has a high runoff potential (i.e., ground-water levels are near the surface).

The results of this analysis can be compared to the flood estimation procedure of Barnes and Golden (1). Their attenuation curve is shown in Fig. 11 in which lakes and swamps attenuate the estimated mean annual flood in a river basin in proportion to the percent of the total basin occupied by lakes and swamps. The curves developed from the present study are also shown in Fig. 11 for comparison. Although not an exact fit, the comparable shape of the curves indicates agreement between these independent analyses.

Summary

The Kissimmee River Basin contributes the major portion of surface inflow and a substantial portion of nutrients to Lake Okeechobee, the primary water supply source for south Florida. With the construction of canal C-38 the basin has changed from one characterized by upland/flood plain retention and slow runoff to one of upland/flood plain drainage and rapid runoff. A basin-wide water management scheme has been proposed based on detention of runoff, restrictions on surface water discharge rates, and routing flow through natural or man-made marshes.

Chandler Slough Marsh has been shown to be moderately effective as a quality and quantity control unit. It is representative of other flood-plain marsh areas within the lower Kissimmee River Basin. Although the results here do not apply directly to upland marshes and sand ponds, it is speculated that these areas would be more effective since they are not subject to as severe a hydrologic loading. By detaining surface runoff, these units will increase detention time and change the regime of much of the runoff from direct to subsurface pathways, thereby, reducing inflow peaks and nutrient concentrations to the downstream marsh.

The flushing of the marsh provides a means by which deposited material (dead vegetation) is removed. By altering the present cycle with an outlet structure,

buildup of material may occur. There is no clear answer as to how long a given marsh can be made to assimilate nutrients.

Acknowledgments

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Local Irrigation Agencies

By Richard L. Revesz[1] and David H. Marks[2]

Introduction

Throughout the Western United States, agricultural areas dependent on irrigation are experiencing limitations on available water supply due to excessive ground-water depletions, to decreases in surface water availability due to interactions with depleted ground water and, in general, to demands for water exceeding readily available supplies. In response to this problem, a considerable amount of engineering literature has been developed on ways to manage and control the distribution of water for irrigation.

In most cases, these have been "supply" models (3,6,7,8,12,15,16,18). Also, these studies view local irrigation agencies as centralized institutions, which make all important decisions concerning the allocation of both surface and ground water in accordance with principles of economic efficiency. As such, they miss two important points: first, farmer demand may well be a function of the types and attributes of supplies available and, thus, policies of demand modification are as important to consider as management alternatives as those of supply increase. Second, because of externalities caused by the actions of individual farmers, institutional mechanisms are needed to implement such plans. During a recent study for the United States Office of Water Research and Technology involving the estimation of farmers response to and acceptance of various policy options to limit water demands, it became clear that problems in the institutional arrangements, necessary to deal with externalities, severely limited the effectiveness of most management models for the conjunctive use of surface and ground water.

These models assume that comprehensive management entities exist. This view is particularly convenient for the study of rational ground water development because it provides a simple method for "internalizing" externalities. These externalities arise from the "common pool" nature of ground water resources, which give a rational irrigator an incentive to pump ground water since the benefits of this water will usually outweigh his share of the adverse consequences

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of aquifer depletion. Naturally, these externalities can be eliminated by giving an irrigation agency the power to enforce an optimal plan for ground-water withdrawals.

The assumption of centralized control is inconsistent with the legal status of ground water as a mineral resource, in some areas, because of the attitudes of fierce independence of individual farmers (1,4,5,9,14,19).

This paper, which presents a historical discussion of the circumstances surrounding the emergence of irrigation institutions in the Western United States, classifies these agencies according to their attributions and reviews their formation processes and modes of operation. In this way, it seeks to provide a realistic framework for future mathematical modeling of the supply of irrigation water. Among the questions that it attempts to answer are:

1. To what extent do local irrigation agencies control surface and groundwater allocations?
2. To what extent do these agencies "conjunctively manage" surface and groundwater resources?

Historical Perspective

In the Western United States, irrigation agencies have been formed for economic and political reasons. In the nineteenth century, economic rationality prompted the formation of water organizations because of the indivisibilities in the physical facilities used to store and transport water. Wells, pipes, canals, and reservoirs exhibit decreasing average costs over a wide range of scales. In general, an individual user can efficiently provide for his own supply only if he transports the water over a relatively small vertical or horizontal distance. Thus, land promoters increased the value of the land they were selling by constructing water systems based on gravity diversion from an adjacent river (4).

During the first half of this century, the formation of large water wholesalers, such as the Water and Power Resource Service (formerly the Bureau of Reclamation), the Army Corps of Engineers and State Department of Water Resources, acted as a catalyst for the emergence of irrigation agencies. Precluded by their charters from dealing with individual farmers, the large water suppliers were forced to sell their water to local agencies, which in turn, distributed it to the farmers in their basin.

In recent years, the major cause for the formation of irrigation agencies has been the rapid decline of ground-water levels. Ground-water depletion results from what hydrologists call "the 'common pool' problem" and is an excellent example of the "tragedy of the commons," a theory formulated to explain the depletion of pastures in medieval England. The common nature of these fields encouraged a rational herdsman to keep as many cattle as possible in them since the benefits he attained from the sale of each additional animal outweighed his share of the negative effects of overgrazing. However, this same conclusion was reached by every rational herdsman sharing the commons and the unrestrained addition of cattle led to the destruction of the fields. Thus, "freedom in the commons brings ruin to all" (13). The "commons" problems of declining water tables include increased operating costs, which vary proportionately with pump lift; large capital costs to deepen wells and install larger

pumps; and intrusion of brine in the aquifers and subsidence is a major problem in the large San Joaquin Valley in California. As much as 4–6 ft, has caused significant problems (i.e., raising of canal linings, building foundations collapsing).

Until very recently, the decline of ground-water levels led to the formation of agencies interested primarily in obtaining additional surface water. In the last few years, agencies charged specifically with the management of ground water have emerged. For example, the Kansas legislature authorized the formation of ground-water management districts in the state on July 1, 1972. Its Nebraska counterpart took the same action on August 23, 1975. However, many states are reluctant to pass laws limiting rights of farmers to pump water on their property.

Thus, local irrigation agencies, charged with the responsibility of supplying water to individual farmers, have arisen for a wide variety of reasons. These institutions may be easily classified into three distinct groups:

(1) Public irrigation districts;
(2) Mutual or cooperative irrigation companies.
(3) Commercial water companies.

Public Irrigation Districts.—Irrigation districts, also known as water conservation districts, water districts, water improvement districts, and conservation districts are government organizations that have legal status as political subdivisions of the state in which they lie (1,10) but, which operate outside the jurisdiction of the established state and local governments.

Public irrigation districts may be formed in two ways: (1) The first is by special state legislation that approves the creation of a district, defines its area, function, organization, and financial authority (5) and (2) the second, possibly only the states that have general enabling legislation permitting the formation of institutions by public request, is through a special petition by local land-owners. The former mechanism has been used extensively in the past, the latter is more popular today.

Where districts are formed by private initiative, an individual, or a group of landowners, presents a petition to the governing body of the county. The petition defines the boundaries of the proposed district, which may only include lands that will benefit directly from the agency's operation, and the proposed sources of water. The petition is accompanied by a list of signatures. Upon receipt of the petition, the county government organizes public hearings in which interested parties discuss the merits of the proposals, and requests a feasibility study from the State engineer. If the approval of the county supervisor is obtained, the county government calls an election in which all the landowners within the boundaries of the proposed district may vote. A major deviation of this rule occurs in California, where the electorate consists of all registered voters (5).

The agency is formally constituted if a majority vote is cast in its favor. Subsequently, a board of directors, an assessor, a tax collector, and a treasurer are elected. In all cases, the newly formed board of directors hires an engineer to prepare a plan of works and estimate their cost. In general, the construction is financed through bond sales, and these have to be approved (usually by a two thirds majority) by the eligible voters in the district. The day-to-day

operation of the agency is carried out by a chief engineer and ditch tenders, and is financed by ad valorem assessments on the land, exclusive of improvements, and through water tolls.

Public irrigation districts belong to a group of organizations designated "special district governments," which include school districts, and housing and port authorities. Labelled the "dark continent of American politics" because of the lack of attention paid to them by political scientists, special districts fill a vacuum created either by the inability of a general government to circumvent constitutional limitations of its tax and debt limit, or by constituent resistance to the expansion of the functional jurisdiction of the state or county. Thus, while a general government is prevented from expanding functionally once it reaches its tax or debt limit, residents of the same area are not legally prevented from organizing a special district that will possess the power to levy taxes or to incur debt, or both (5).

An 1865 law in the Territory of Utah was the first legislative act authorizing the creation of irrigation districts in the United States. A similar act was passed in California in 1872, and between that date and 1887, special acts of the state legislature created a large number of other districts (19). The formation of special irrigation districts in California received a big boost with the passage of the Wright Act in 1887. This act permitted citizens of towns having 500 or more electors to propose and approve by election the formation of an irrigation district. The proponents of this measure hoped that a districts's assessments would finance irrigation works and, at the same time, produce the subdivision of large land-holdings into relatively small tracts (4). In 1896, after a decade of litigation in state and federal courts, the U.S. Supreme Court upheld the constitutionality of this law.

This decision paved the way for the formation of numerous irrigation districts in the Western United States. While, at the turn of the century, only California and Washington had active districts of this type, Nebraska, Colorado, Oregon, and Idaho adopted them during the first decade of this century. In 1950, 483 local public irrigation districts were supplying water in 17 of the 19 western states (1,5).

Public water agencies supplying surface water possess the following powers (4):

1. Corporate powers include the rights to sign contracts, construct works, buy and sell water, operate irrigation works, and sue and be sued.
2. The power of eminent domain is subject to certain broad guidelines. They can condemn property needed for designated uses.
3. The power to incur bond indebtedness allows agencies to issue general obligation bonds, or revenue bonds, or both. In all cases, the floating of a bond issue must be approved at an election by the member landowners or eligible voters.
4. The power to fix assessments on their constituents secure and retire bond indebtedness and cover operating expenses. In some cases, the legal uses of these revenues are limited by state laws. In other cases, the establishment of assessments may require the approval of the electorate.
5. The power to establish charges provides for establishment of tolls to pay for the delivery of water or for other services.

6. The power to purchase water permits agencies to enter into short-term contracts with local "overlay" agencies or with the large water supplying agencies, such as the Army Corps of Engineers and state departments of water resources. In general, they may not sell water outside the jurisdiction of their districts. However, they may sell "surplus water" through short-term contracts.

The powers allow public districts concerned primarily with the supply of surface water to have an impact on ground-water depletion. A land assessment imposes a fixed cost on farmers. Thus, the variable cost of surface water, collected through water charges, is less then the total cost of securing this water. If the cost of pumping a comparable amount of ground water fell between the variable and total cost of surface water, an economically rational farmer would prefer surface to ground water, even though, in the absence of a district, the ground water would have been less expensive.

Public irrigation districts that supply water may slow down the process of ground-water depletion by securing additional surface water or by setting high land assessments and low user charges on the water they sell. However, with certain exceptions, they cannot deal directly with the problem of falling aquifer levels. In some states, the legislatures take an active role in protecting ground-water resources. For example, in 1957, the Nebraska legislature prohibited the location of one irrigation well within 600 ft of another and mandated that irrigation wells be registered within 15 days of their completion (17). More recently, the Kansas and Nebraska legislatures have authorized the formation of public ground-water management districts.

In general, these districts have the following powers:

1. Metering to install or to require the installation of meters and to read or to require water users to read these meters.
2. Set standards to adopt and enforce reasonable standards for the conservation of groundwater within the district.
3. Ensure compliance to enter private property to determine conformance with established rules in the use of water.

In addition, these organizations possess most of the power assigned to irrigation districts that supply surface water.

Mutual Irrigation Companies.—Mutual companies are the most common type of irrigation enterprise in the West. In 1950, there were 9,200 cooperative irrigation enterprises in the western states, of which the largest number (2,265) was in Colorado, and the next largest (1,270) was in California. The United States Irrigation Census gives all these companies the classification "mutual." However, they are known by various designations, including "mutual water companies," mutual irrigation companies, cooperative irrigation companies, mutual canal companies, and mutual ditch companies (1). A mutual company is a voluntary, nonprofit enterprise engaged primarily in providing water for its shareholders.

Although there are characteristics common to most mutual companies, these enterprises differ considerably from state to state. To a great extent, the existing variations may be ascribed to the historical conditions surrounding their emergence.

In Colorado, the mutual companies were organized by the holders of water rights gained by prior appropriation. Thus, shareholders are allowed to use the water to which they are entitled on any land served by the irrigation system. This feature of the state's mutual companies gives its Colorado shareholders the right to sell surplus water to other shareholders in the same basin. This practice, known as "renting," is described in detail by Anderson (2).

In Idaho, many mutual companies were organized under the provisions of the Carey Act of 1894. This act allocated funds for the reclamation of public desert land and authorized the states to contract with private construction companies for building irrigation works. These companies were then able to sell "water rights" to the settlers. The Carey Act stipulated that mutual companies would eventually operate the irrigation system. It also made water right appurtenant to the land, thus, precluding intrabasin transfers. In contrast, in central California, mutual companies were formed without federal supervision. In this region, water can be sold or leased for use in any land that may be irrigated in the system (1).

In New Mexico, mutual companies incorporate practices brought to the southwest by the Spanish. For example, under New Mexico's law, ditches that are neither private nor incorporated but have more than two owners are "community acequias" in the absence of an agreement to the contrary.

A mutual company can be either unincorporated or incorporated. The former, in its simplest form, is merely an informal agreement among a small group of neighboring farmers to operate jointly irrigation facilities. More formal arrangements can be made under "articles of association" that set up a company organization with bylaws to govern the rights and obligations of its members and the management and operation of irrigation facilities.

In 1950, 69% of the irrigation cooperatives operating in the western states were unincorporated. Of these, the largest number (1,957) was in Colorado; the next largest group (994) was in Montana. The widespread popularity of unincorporated companies attests to the fact that institutions based entirely on cooperation among irrigators can operate successfully.

However, unincorporated mutual companies face many disadvantages. In particular: (1) The members of an unincorporated company are joint owners of the irrigation works, and all partners must consent unanimously to incurring debts, executing contracts; (2) individual members may be held liable for the debts of the enterprise; and (3) there is no practical way of compelling members to contribute to the operating costs.

These disadvantages have led to the incorporation of many mutual companies. This procedure consists, in general, of filing "articles of incorporation" with the appropriate state agency. The enterprise then becomes a "body corporate," with the authority to hold property in the corporate name and to exercise the powers given to corporations in that state. The "articles of incorporation" also specify the following powers to be exercised by the corporation: number and terms of office of its governing body and number of shares of capital stock. They also give the board of directors the authority to enforce regulations concerning delivery of water, care and operation of the irrigation system, time and method of payment for water and duties of the operating employees.

The main benefits that mutual irrigation companies gain through incorporation are the following:

1. The corporation, as a legal entity, can act in its corporate name without first obtaining the consent of all its members.
2. The board of directors, elected by the shareholders, has the authority to conduct the affairs of the enterprise and to enforce the payment of obligations by the shareholders.
3. The company is in a better position to borrow money because it can pledge the assets of the company as collateral.

Unlike public irrigation districts, mutual companies cannot float bonds, do not have the power of eminent domain, and cannot levy land assessments. They are not legally eligible to purchase water from federal and many state wholesalers. Thus, they depend mostly on their own integrated supply. Also, in general, they may not sell water to the outside public. The latter impediment often arises from charter provisions (and state water rights' doctrines) that makes water rights appurtenant to specific parcels of land held by the shareholders. However, even when provisions of this type are not present, the outside sales become subject to the limitations imposed on privately owned public utilities (4). In fact, if outside sales become important, the mutual company may risk being declared a public utility and, thus, become subject to the set of regulations discussed in the next section.

Commercial Water Companies.—The mechanisms that govern the operation of commercial water companies are relatively simple: individual proprietorships operate, as such, under the general state laws while privately owned public utilities engaged in the water industry are established under federal and state laws governing private corporations.

Privately owned public utilities are especially restricted in that they must acquire franchises to provide services to defined areas and cannot refuse to serve customers in those areas. Also, the rates they charge for the water they deliver are regulated by the public utilities commission of the state in which they operate. However, unlike mutual companies, they have the power of eminent domain and can issue bonds (1,4).

Historically, commercial water companies have played an important role in the development of irrigation, as was discussed before private irrigation companies emerged in the latter part of the nineteenth century, usually in conjunction with land promotions. However, at present, commercial companies do not play a prominent part in the development and distribution of irrigation water.

As Bain, et al., points out (4), it would appear that commercial water companies, natural monopolies facing a demand that is relatively inelastic in the short run, should be successful. However, they exhibit numerous weaknesses:

1. Although they are the only feasible suppliers of water to the farmers, they are also its only customers.
2. The water companies are often caught between customers who insist upon regular service and sources of supply that are naturally uncertain.
3. The state public utilities commissions are often hostile in setting the water rates.
4. Their limited legal powers (compared to those of public districts) often lead to their replacement by stronger agencies when their customers decide to expand their water supply systems.

For these reasons, commercial water companies have almost disappeared (1). For example, in California, only one private irrigation company has survived (4).

Summary and Conclusions

Table 1 summarizes the major characteristics of irrigation enterprises and shows the major differences among the principal types of agencies. Most important, it shows that these agencies differ in the way they finance their operations, obtain water supplies and control ground-water withdrawals. Thus, studies which group all agencies into one class and give these the power to

TABLE 1.—Major Characteristics of Irrigation Enterprises

(1)	Public district (2)	Mutual company[d] (3)	Commercial company (4)
Formation	special legislative act or landowner petition	articles of incorporation	articles of incorporation
Management	board of directors	board of directors	board of directors
Voting	all landowners[a]	shareholders	shareholders
Outside regulation	none[b]	none[b,c]	state public utilities commission
Power of eminent domain	Public agencies	none	public agencies
Bond issues	government bonds	none	corporate bonds
Land assessments	yes	no	no
Water charges	yes	yes	yes
Profit making	no	no	yes
Supplies of water	own supplies, state, and federal wholesalers	own supplies	own supplies
Direct regulation of groundwater withdrawals	groundwater management districts only	no	no

[a]In California, all waters.
[b]Must comply with general State and Federal laws.
[c]As long as no important sales to nonmembers take place.
[d]Incorporated.

control all surface and ground-water releases provide an unrealistic oversimplification of the actual process of distribution of irrigation water. For example, only public irrigation districts which also control ground-water withdrawals—a small fraction of all irrigation agencies—have the powers to engage in the actions envisaged by these studies. Other public irrigation districts can only control ground-water withdrawals indirectly: by increasing the supply of surface water or by pricing techniques. Mutual and commercial companies may do so only by increasing the supply of surface water.

This conclusion pinpoints the weakness of the phrase "conjunctive management

of surface and groundwater." Indeed, as long as groundwater diversions are unrestricted, the conjunctive nature of the management is very tenuous, since most basin managers may affect only the rate of ground water by indirect means. Thus, conjunctive use "modelers" should modify their proposed objective functions to include only those decision variables controlled by the agency under study.

This paper answers the two questions posed in the introduction, but leaves two important questions partially unanswered:

1. What are the ideal powers of an irrigation enterprise?
2. How should an irrigation agency attempt to alter the water demands of individual farmers?

The first question raises the problem of the tradeoff between efficiency and local control. In general, the operation of water distribution will be more efficient when the irrigation agency controls both surface and ground-water releases. However, the granting of additional powers to these agencies often conflicts with the objective of local control over the distribution system and with the individualism of farmers. This tradeoff could form the focus for a future study on this subject. The second question is clearly a question for further research.

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Operation Policy Analysis: Sam Rayburn Reservoir

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Setting

The Sam Rayburn Reservoir System includes two reservoirs in series: Sam Rayburn Reservoir on the Angelina River and B. A. Steinhagen Lake and Town Bluff Dam, known as Dam B Reservoir, on the Neches River in eastern Texas. These reservoirs are operated by the U.S. Army Corps of Engineers. The system components are shown in Fig. 1.

Operation of Sam Rayburn Reservoir provides flood control, power generation, water supply, water quality maintenance, and recreation. Runoff from approximately 3,449 sq mi (8,940 km^2) drains into the reservoir. The total storage volume of the reservoir is 561,000 acre-ft (691,713,000 m^3); 289,600 acre-ft (357,076,800 m^3) of the volume are allocated to conservation purposes, and the remainder is allocated to flood control. The installed capacity of the two hydropower units at the reservoir is 52,000 kW, and the "dependable" capacity currently is estimated to be 49,000 kW.

Dam B was constructed and is operated primarily for reregulation of releases from Rayburn Reservoir. The reservoir is operated also for water supply and for recreation. Total storage available is 306,400 acre-ft (377,791,200 m^3).

Additional detailed information on Sam Rayburn Reservoir is presented in Ref. 8. Information on B. A. Steinhagen Lake and Town Bluff Dam is available in Ref. 6.

Current Operation Problems

Due to the proximity of the reservoir system to the Gulf of Mexico, maintenance of sufficient discharge downstream of Dam B is critical to prevention of saltwater

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intrusion. This intrusion is detrimental because water is withdrawn from the Neches River for irrigation and for municipal and industrial water supply. The average maximum monthly discharge rates for recent years are shown in Table 1. Historically, a saltwater barrier has been installed downstream from Dam B during periods of little runoff because releases are reduced during these periods. With such a barrier in place, the downstream discharge requirement is reduced by approximately 1,000 cfs (28 m^3/s) because the need for water to prevent saltwater intrusion is eliminated. Subsequent discharge that exceeds the demand

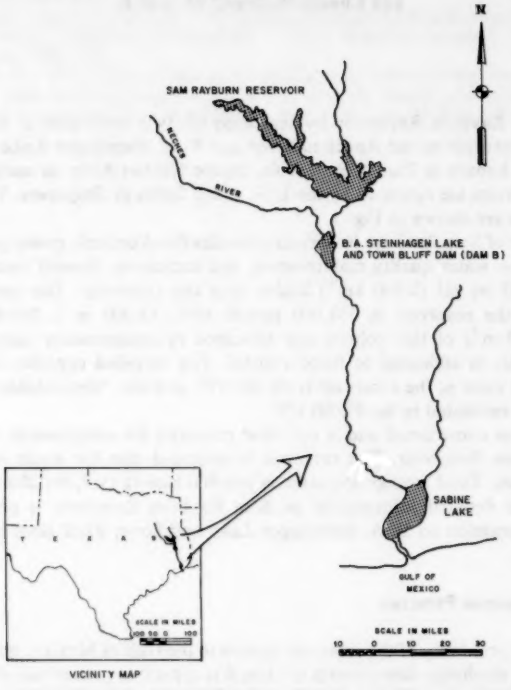

FIG. 1.—Sam Rayburn Reservoir System Components

by approximately 2,000 cfs (56 m^3/s) causes the barrier to be "washed-out." Thus one of the operation problems is to select an operation policy that minimizes the number of times that a saltwater barrier must be installed.

Selection of operation rules that will yield the optimal hydropower production from Rayburn Reservoir is the second operation problem considered. The minimum acceptable energy output is defined in a contract between the Sam Rayburn Dam Electric Cooperative, Inc., and the Federal Government. The contract states:

> . . . the government agrees, to the extent that water is available in the McGee Bend Reservoir (now Sam Rayburn Reservoir) above elevation 149, to make releases . . . as required for the generation of power, with such releases at least sufficient to generate power equivalent to 42,000 kilowatts for a minimum period of 75 hours per month for each of the six monthly periods from mid-April through mid-October of each year (8).

Additional useable power often can be generated, and, if so, is purchased by a private utility. Thus a dependable power output must be defined, and operating rules must be selected to provide the power with high reliability. The operation rules also should yield as much additional useable power as possible.

The facilities for recreation at Rayburn Reservoir and at Dam B pose another operation constraint: the reservoirs should be operated in such a way that the pool elevation fluctuations are not intolerable to those using the facilities.

TABLE 1.—Water Supply Demand Schedule: Sam Rayburn Reservoir System

Month (1)	Average maximum monthly demand, in cubic feet per second* (2)
January	250
February	300
March	700
April	1,100
May	1,400
June	1,700
July	1,400
August	800
September	450
October	350
November	300
December	250

*1 cfs = 0.028 m^3/s.

In addition to other previously mentioned purposes, Sam Rayburn Reservoir and Dam B Reservoir are regulated to provide flood control downstream. The channel capacity downstream of Dam B is approximately 20,000 cfs (560 m^3/s), so the reservoirs are operated to maintain flows at or below this capacity if possible.

Solution Methodology

A combined simulation-optimization approach is employed to select an optimal operation policy for Sam Rayburn Reservoir System. This methodology is shown schematically in Fig. 2. The simulation model is a single reservoir model that accounts for water use throughout the system, satisfying all demands when possible and allocating the available water according to specific priorities when

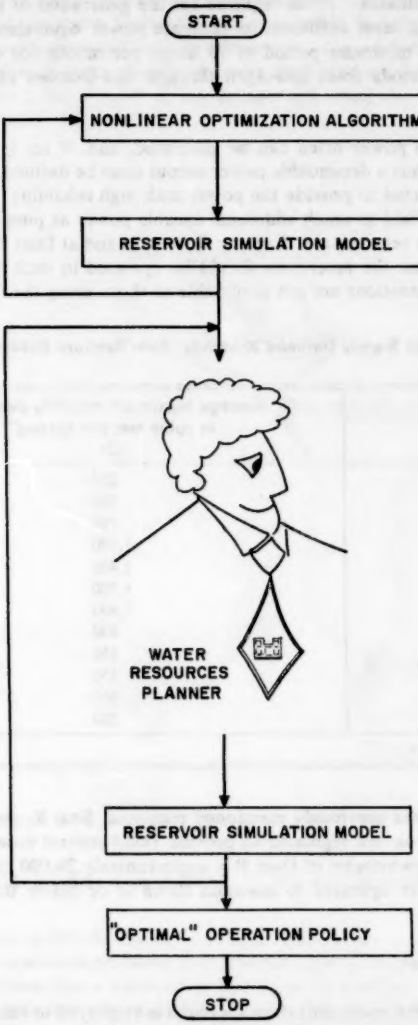

FIG. 2.—Schematic of Solution Methodology

conflicts exist. The simulation model is linked with a nonlinear programming algorithm that selects automatically the optimal operation policy for the reservoir system for the given data and with a user-specified objective function. A weighted combination of 10 indices of operation efficiency can be used to define this objective function. The operation policy that is identified as the optimal policy by the nonlinear programming algorithm is smoothed using engineering judgment based on experience with operation of the system, and system response with this smoothed policy is simulated. This step is repreated as necessary to obtain an acceptable operation policy. The general approach was suggested by Jacoby and Loucks (11).

Alternative techniques for selection of an optimal allocation of available storage have been proposed and were considered, including applications of linear programming (13), network flow programming (14,15) and dynamic programming (1,4,16). However, as Yeh et al. (16) point out, ". . . there appears to exist no general algorithm." Each application of these mathematical programming techniques has required some development and research to select and to program the most efficient solution procedure. In this study, time constraints and budget limitations precluded such research and development, so a readily available, generalized simulation program was combined with readily available computer code for the nonlinear programming algorithm (12). This approach provides the important capability to simulate in detail the operation of reservoir system with a model that can easily be used independent of this optimization algorithm.

Simulation Model.—The operation of the Sam Rayburn Reservoir System is modeled with the Reservoir Yield Program developed in the Hydrologic Engineering Center (HEC), with modifications to simulate accurately the operation of this particular system and to model the format of the operation policy traditionally used with this reservoir system. The Reservoir Yield Program simulates the conservation operation of a reservoir system that includes one reservoir and one downstream control point. Constraints on discharge can be specified at the reservoir and at the control point. The model is designed for analysis of operation with a long time interval, such as one month. The methods of computation in the Reservoir Yield Program follow closely the procedures traditionally employed in hand computations. For each computation period, the reservoir release equals the maximum minimum flow requirement for all system purposes unless this conflicts with maximum permissible flows. In that case, the reservoir release is restricted to the minimum maximum permissible flow. Absolute control over the release is exercised by full reservoir and empty reservoir limitations. Power is assumed to be generated from reservoir releases up to plant capacity, with power head determined by successive approximations to account for variation of head with discharge. Flows are translated from the reservoir outlet to the downstream control point in a single period without routing. Further detailed description of the methods of computation employed in the Reservoir Yield Program is presented in Ref. 7.

Operation of the Rayburn System can be simulated adequately for the purposes of the study using the Reservoir Yield Program with a monthly computation interval because Dam B has no significant monthly carry-over storage capacity. Dam B can be represented as a control point, with average monthly outflow considered equal to average monthly inflow, and all water requirements downstream of Dam B can be modeled as requirements at the control point.

Modifications to the Yield Program required for simulation of the operation of the Rayburn System include the following: (1) Modifications to employ a storage level concept in operation of Rayburn Reservoir; (2) modifications to reflect the installation of a downstream saltwater barrier when the volume of water in storage in Rayburn falls below a specified value; (3) modifications to allow specification of power requirements and downstream discharge requirements as a function of storage in Rayburn; and (4) modifications to alter the system operation goals so releases required to satisfy minimum power generation requirement at Rayburn will have highest priority as required by contract.

Use of storage levels for specification of the operation rules for Rayburn Reservoir is accepted practice at that reservoir, so modification of the program to employ the levels is necessary if practicable operation rules are to be selected. Incorporation of storage levels for operation of Rayburn is accomplished by defining the conservation storage volume allocated to each of four imaginary zones illustrated in Fig. 3. At the beginning of each period of simulation, the

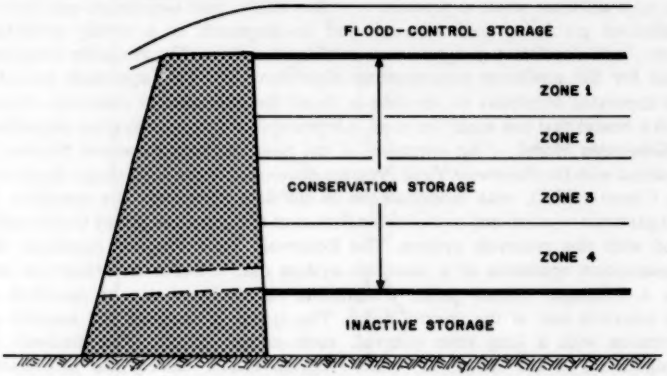

FIG. 3.—Imaginary Reservoir Storage Zones

current level is determined by comparing the beginning-of-period storage value with these bounds, and the at-site power requirements and downstream discharge requirements are set, as shown in Table 2. The Reservoir Yield Program is executed as before.

Additional modifications to the Reservoir Yield Program provide for simulation of installation and failure of a saltwater barrier downstream from Dam B. Installation of the barrier is assumed to occur when storage in Rayburn Reservoir falls to Level 3 or 4 and remains in either level for three months (thus simulating a time lag for decision and for installation of the barrier). When the barrier is not in place, downstream discharge targets are increased to prevent saltwater intrusion, as shown in Table 2. When the barrier is installed, the targets are fixed at the actual water supply demand until the barrier is "washed-out" by excessive discharge. This excessive discharge is defined as 2,000 cfs (56 m^3/s) or the downstream requirement plus 1,000 cfs (28 m^3/s), whichever is larger.

To model adequately the priorities of releases in the Rayburn System (priorities which are contrary to those incorporated in the Yield Program), the algorithm that selects the release for each period is modified to give highest priority

TABLE 2.—Power and Discharge Requirements: Sam Rayburn Reservoir System

Level[a] (1)	Power requirement at Rayburn Reservoir (2)	Discharge requirement below dam (3)
1	20% plant factor	Water supply demand + flow to prevent saltwater intrusion
2	75 hr of generation (approx 10% plant factor)	Water supply demand + flow to prevent saltwater intrusion
3	75 hr of generation 15 April–15 October No requirement in other months	Water supply demand
4	75 hr of generation 15 April–15 October No requirements in other months	No specific operation requirement; shortages declared if discharge fails to meet demands of Level 3

[a]Note that the convention of numbering levels for this study does not correspond to the convention in other reservoir simulation programs developed in the Hydrology Engineering Center (9,10).

TABLE 3.—Possible Objective Function Terms for Sam Rayburn Reservoir Operation Analysis

Function (1)	Description (2)
1	Energy shortage index[a,b]
2	Downstream discharge shortage index[a]
3	Number of times saltwater barrier is installed in period of analysis
4	Number of times saltwater barrier fails (is washed-out) in period of analysis
5	Average annual energy shortage[b]
6	Average annual downstream discharge shortage
7	Average monthly conservation pool elevation fluctuation
8	Average annual energy
9	Number of times conservation pool is emptied
10	Number of times downstream discharge shortage occurs

[a]Each shortage index is computed by summing the squares of the annual shortage ratios and multiplying by (100/number of years of analysis). The annual shortage ratio is expressed as the ratio of the annual shortage divided by the annual requirement.

[b]Energy shortage is equivalent to "power" shortage computed by the Reservoir Yield Program. For this study, shortage is defined as follows: Shortage = maximum (0., Level 1 energy requirement − energy generated).

to satisfaction of the minimum power requirement at Rayburn Reservoir, as required by contract. With the modification, releases necessary to generate the required power, rather than certain minimum flow requirements, are given first priority.

TABLE 4.—Summary of Selected Operation Efficiency

Operation objective (1)	Plant factor, as a percentage (2)	Downstream demand schedule (3)	Discharge to prevent saltwater intrusion, cubic feet per second (4)	Number of times saltwater barrier is installed (5)
Maximize average annual energy	20	Maximum	1,000	57
	20	Average	1,000	49
	25	Maximum	1,000	57
Minimize energy shortage	20	Maximum	1,000	25
	20	Average	1,000	13
	25	Maximum	1,000	20
Minimize barrier installation	20	Maximum	1,000	8
	20	Average	1,000	1
Compromise	20	Maximum	1,000	14
	20	Average	1,000	6
	20	Average	1,200	6

[a]Values shown are for 51-year analysis period.
Note: 1 cfs = 0.028 m^3/s; 1 ft = 0.305 m.

Optimization Model.—To determine the optimal operation policy for the Sam Rayburn System, the reservoir operation problem is formulated as a constrained nonlinear programming (NLP) problem. The decision variables in this optimization problem are the volumes of conservation storage to be allocated to each of the four operation levels. These decision variables are subject to upper and lower bounds; the volume allocated to each level must be greater than zero and must not exceed the total volume of conservation storage available. Also, the sum of the volumes allocated to the four levels must equal the total conservation storage available. The storage allocation currently varies seasonally, with seasons defined on the basis of significant change in rainfall pattern as follows: (1) March–April; (2) May–June; (3) July–September; and (4) October–February.

The optimization problem may be expressed mathematically as

$$\text{minimize} \quad f(X) \qquad (1)$$

$$\text{subject to} \quad 0 \leq x_{ij} \leq \text{STMX} - \text{STMN} \qquad (2)$$

$$\sum_{i=1}^{4} x_{ij} = \text{STMX} - \text{STMN}; \quad j = 1, 2, 3, 4 \qquad (3)$$

in which X = a vector of all decision variables x_{ij}; i = the index of storage levels; j = the index of seasons; STMX is the total storage volume at the top of the conservation pool; STMN is the total storage volume at the bottom of the conservation pool. If desired, this formulation can be modified to allow monthly variation of the storage allocation. The objective function, $f(X)$, is

Indices: Sam Rayburn Reservoir System*

Average annual energy shortage, thousand kilowatts-hours (6)	Average monthly conservation pool elevation fluctuation, in feet (7)	Average annual energy, thousand kilowatt-hours (8)	Number of times conservation pool is emptied (9)	Number of months in which downstream discharge shortage occurs (10)
34,264	.69	115,073	0	110
33,120	.56	115,072	0	32
47,634	.69	115,095	0	114
15,735	1.03	113,632	31	32
13,007	.93	113,971	31	17
27,624	1.10	112,689	60	58
25,670	1.01	113,529	19	31
25,856	.73	114,467	4	12
26,656	.93	114,250	0	28
24,857	.75	114,570	0	9
23,356	.81	114,411	0	4

evaluated by executing the modified Reservoir Yield Program with specified values of the decision variables.

The Box-Complex algorithm (3) is employed to solve the constrained nonlinear programming problem. This algorithm is a multivariate, constrained, random-search technique that seeks the minimum (or maximum) of a general nonlinear function subject to explicit upper and lower bounds on the decision variables (Eq. 2) and to nonlinear constraints on the decision variables (Eq. 3). With the Box-Complex algorithm, a set of feasible solutions to the optimization problem is generated at random, the objective function is evaluated for each, the "worst" solution is discarded, a new solution is determined with a projection technique, and the process is repeated until convergence criteria are satisfied.

Multiple Objective Analysis.—The efficiency of operation of the Sam Rayburn Reservoir System cannot be measured solely in economic terms, in terms of power generation, or in terms of failure to satisfy discharge requirements. These and other indices of operation efficiency, must be considered when selecting the optimal operating policy, and the trade-offs must be considered when selecting the optimal policy. For example, if the storage is allocated to maximize the average annual energy generated, the number of times that the saltwater barrier must be installed may be unacceptable. On the other hand, if storage is allocated to minimize the number of times the barrier must be installed, the energy generated decreases and may fall below an acceptable level. Neither solution is likely to be acceptable in terms of overall system operation goals, so some compromise solution must be selected.

A weighting method of multiobjective programming is employed to quantify the relative importance of various operation objectives (5). With this technique,

the mathematical objective function for the NLP problem is defined as

$$f(X) = \sum_{k=1}^{p} w_k z_k(X) \quad \ldots\ldots\ldots\

the effectiveness of each. These results are also shown in Table 4. Fig. 4 illustrates the storage allocation of one of these operation policies.

Conclusions

From the perspective of the water resources planner, the most important conclusion that may be drawn from this study is that certain analytical techniques presented in the literature are applicable to practical resource management problems. The Sam Rayburn Reservoir operation problem is solved as a nonlinear programming problem, using an accepted simulation model to evaluate the objective function for each set of operation rules. The nonlinear programming algorithm employed is a simple, readily available technique. A multiobjective programming technique is used to develop an objective function that, in some sense, quantifies the importance of various system purposes.

From the perspective of the water resources system analyst, two important conclusions may be drawn from this study. The first is that planners and engineers involved in planning and managing water resources projects will accept application of systems analysis techniques to problems they face if such applications can be demonstrated to: (1) Provide additional information for use in decision making; (2) reduce the time, money, or computer memory requirements for plan formulation or evaluation; or (3) increase the project benefits by identifying solutions that satisfy the practical constraints on operation and are sufficiently resilient to respond to changing conditions. Integrated use of a nonlinear programming formulation with the Reservoir Yield Program for simulation of system operation, followed by an interactive smoothing process that allows input from the water managers satisfies these requirements.

In the process of developing operation rules for Sam Rayburn Reservoir, Corps personnel who are involved daily with the operation were consulted in definition of the problem, in identification of the critical characteristics of the system that should be modeled, and in evaluation of the solutions developed by application of the optimization-simulation methodology. The results of the initial simulations of system operations were reviewed carefully by Corps District and Division personnel to assure that the modified reservoir simulation program adequately modeled the system operation. This leads logically to the second conclusion: the resource managers/system operators must be included in the policy formulation-evaluation "DO-Loop" at many points.

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Appendix II.—Notation

The following symbols are used in this paper:

f = total objective function;
p = total number of objective functions;
STMN = total storage volume at bottom of conservation pool;
STMX = total storage volume at top of conservation pool;
w = weight assigned to objective functions;
x = decision variable;
X = vector of all decision variables; and
z = objective function.

Subscripts

i = index of reservoir conservation storage levels;
j = index of seasons; and
k = index of objective functions.

INTERACTIVE NONSTRUCTURAL FLOOD-CONTROL PLANNING

By David T. Ford,[1] A. M. ASCE

ABSTRACT

The practical success of any flood control plan depends on the ability of the planner to analyze the available data to formulate a feasible plan, considering systematically the many possible alternative solutions. In planning for nonstructural flood-control measures, much of the pertinent data is descriptive of the structures in the flood plain. As the number of structures increases, the complexity of the data management problem increases substantially, and a search through the data to identify viable alternatives consumes resources that are better allocated to analysis of social acceptability, environmental impacts, and other factors that affect plan feasibility. Furthermore, the arithmetic required to evaluate a multitude of alternatives for individual structures is time-consuming, so that the number of alternatives that are actually considered for each structure is likely to be reduced as the number of structures increases.

Computer programs of the Nonstructural Analysis Package, developed in the Hydrologic Engineering Center (HEC), obviate the data storage, manipulation, and retrieval problems and provide for rapid evaluation of alternative nonstructural flood-control plans. The Nonstructural Analysis Package was used in development of a proposed flood-control plan for Santa Fe, New Mexico.

Nonstructural Flood-Control Measures.—Nonstructural flood-control measures mitigate flood damages by modifying damage susceptibility of property in the flood plain or by modifying the loss burden. In contrast, the more well-known structural flood-control measures, such as dams, floodwalls, and levees, mitigate damages by modifying the characteristics of the flood. Nonstructural flood-control measures are further characterized by the relative simplicity of design and construction technology, and by the small scale, local level at which these measures reduce the flood hazard. The following is a list of "nonstructural" flood-control measures:

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1. Installation of temporary or permanent closures for openings in structures.
2. Raising existing structures in-place.
3. Constructing new structures on fill or columns.
4. Constructing small walls or levees around structures.
5. Relocating or protecting damageable property within an existing structure.
6. Relocating existing structures or contents, or both, out of a flood hazard area.
7. Use of water resistant materials in new or existing structures.
8. Regulation of development of flood-plain land by zoning ordinances, subdivision regulations and building codes.
9. Acquisition of title or easement to flood plain land.
10. Flood insurance.
11. Installation of flood forecast and warning systems with an appropriate evacuation plan.
12. Adoption of tax incentives to encourage wise use of flood plain land.
13. Placement of warning signs in the flood plain to discourage development.
14. Adoption of development policies for facilities in or near flood plain land (2).

Planning Nonstructural Flood-Control Measures.—Development of acceptable flood-control measures is best accomplished as an iterative process in which the water resources planner: (1) Evaluates the flood hazard in the flood plain; (2) formulates a physically feasible, socially and politically acceptable measure; (3) evaluates the efficiency of the measure in terms of the multiple objectives of water resources planning; and (4) repeats steps 2 and 3, as necessary. To accomplish these steps, the planner must collect, process, store, retrieve, manipulate, and integrate hydrologic, hydraulic engineering, economic, social suitability, and community well-being information (1).

Methodologies for planning nonstructural flood-control measures are necessarily different from those employed for planning structural flood-control measures. Because of the broad protection provided by structural measures and of the manner in which this protection is provided, the characteristics of individual structures, such as depth-damage functions, can be aggregated for analyses, and the efficiency of proposed measures can be evaluated relative to the aggregated functions. On the other hand, most nonstructural measures affect a single structure or small group of structures. Consequently, nonstructural flood control plans should be formulated on a structure-by-structure basis, and the efficiency of proposed plans is logically evaluated in that same manner.

The existing flood hazard for structures in the flood plain is evaluated with hydrologic, hydraulic, engineering, and economic information, which is used to delineate areas subject to inundation and to estimate flood flows and the severity of floods of varying frequency. Indices commonly used to quantify the hazard include: (1) Depth of flooding due to an historical or hypothetical flood; (2) damage incurred by an historical or hypothetical flood; (3) frequency of flooding to some specified elevation; (4) level of protection; and (5) the expected annual value of flood damages. The first index is determined by ascertaining the discharge associated with the flood of interest and then by referring to a discharge-depth function to establish the corresponding depth. The second index is evaluated by carrying the process one additional step to

determine the damage associated with the depth. Evaluation of the third index requires development of an elevation-frequency function; the frequency of flooding to a specified elevation can be determined by referring to this function. The fourth index, level of protection, is defined as the recurrence interval (reciprocal of probability) of the maximum nondamaging flood event for a particular structure. Evaluation of level of protection requires analysis of an elevation-damage function to define the elevation at which damages start, followed by analysis of the elevation-frequency function to determine the corresponding recurrence interval. The final index, the expected annual flood damage, is defined as the average damage that can be expected in a year and is computed by summing the products of the damage caused by each depth of flooding and the probability of each of those depths occurring.

Formulation of successful nonstructural flood-control plans requires integration of information on the flood hazard, the engineering characteristics of the flood plain structures, the environmental, the social suitability and the community well-being. Clearly the flood hazard, engineering, and environmental information are important, but the social and political suitability of a proposed plan will ultimately dictate the acceptability of the plan. Consequently, this information is properly considered early in the formulation and provides the initial screening criteria. For example, if the citizens of a community express through public meetings a strong opposition to certain flood-control measures, these measures can be eliminated from consideration. As measures that are socially and politically feasible are identified, a task that is best accomplished on-site, the flood hazard, engineering, and environmental information is employed for further screening. This approach differs from the more commonly followed approach that applies first economic screening criteria in the formulation step, followed by social and political suitability criteria. The process is iterative; the planner must have a perception of the engineering and economic effects of various measures and must combine this perception with the knowledge of political and social suitability at each step in the planning process.

Data Management Problem

Problems of data management exist at all steps of nonstructural flood-control planning. Evaluation of the existing hazard requires management of depth-damage, discharge-depth, and discharge-frequency functions, along with certain engineering data description of the location of structures in the flood plain. Formulation of alternatives requires management of additional data descriptive of flood plain structures and management of social, political, and environmental information that defines constraints on project selection. Evaluation of proposed measures necessitates management of much of the same data required for assessment of the existing flood hazard and of additional information required for environmental impact assessment. For a flood plain that contains hundreds of structures, the data management task can be monumental and can consume manpower resources that are properly allocated to other planning activities.

As an example of the magnitude of the data management problem, consider the task of identifying those structures for which small walls might be feasible in a flood plain containing 500 structures. Johnson (2) suggests that small walls are generally feasible for single-story structures without basements that are

located below the level of the 15-year flood. Identification of the structures for which this measure would be feasible requires data manipulation of the 500 structures to compute the level of protection of each, comparison to determine which lie within the 15-year flood plain, retrieval of structure-type descriptor for these structures, and further comparison to determine which are single-story with no basement. Evaluation of the economic benefits of the small walls would require further retrieval, modification, and manipulation of the functions, and recomputation of the expected annual damages for the structures. Several thousand data elements must be managed to complete this task. Additional iterations with alternative measures will require further manipulation of the data.

Solution to Data Management Problem

Nonstructural Analysis Package.—The computer programs of the Nonstructural Analysis Package were developed to manage large amounts of data, which may be necessary for nonstructural planning, and to perform the calculations which may be necessary to assess the efficiency of alternative measures, thereby freeing the planner to think. The programs focus on individual structures or on userdefined groups of structures as the elements of analysis. Alternative nonstructural flood control measures are evaluated on a structure-by-structure basis, just as these measures would be implemented.

The Nonstructural Analysis Package includes two computer programs that accomplish the following tasks: (1) Read data describing the hydrologic, hydraulic, and physical characteristics of a river channel and of the surrounding flood plain; (2) read engineering data describing structures within the flood plain; (3) merge the data to develop information on the flood hazard at each structure using a technique described by Webb and Burnham (6); (4) prepare a computer file containing the data collected and the resulting information on the flood hazard; (5) selectively access and display the basic data and the indices of flood hazard; and (6) modify the data and recompute the hazard indices to reflect the effects of proposed nonstructural flood-control measures. Tasks 1–4 are accomplished by a computer program designated the Preprocessor; tasks 5 and 6 are accomplished by a program designated the Interactive Analysis Program.

Preprocessor Program.—The Preprocessor Program manipulates basic hydrologic, hydraulic, economic, and engineering data to derive the indices that quantify the existing flood hazard at each structure (or group of structures) and to arrange the information in a data bank so the efficiency of alternative measures can be evaluated. This manipulation is shown schematically in Fig. 1. The objective of the manipulation is the derivation of the elevation-frequency and damage-frequency curves for each structure or group of structures. Typically, frequency information is developed only at a stream gage location, so the information must be transferred to each structure. This is accomplished by first combining the discharge-frequency function with an elevation-discharge relationship to derive an elevation-frequency function. Then using information on the hydraulic characteristics of the stream and the surrounding flood plain, elevations at a point on the stream can be related to elevations at a structure, and an elevation-frequency function for that structure can be derived. If this function is combined

with the elevation-damage relationship for the structure, the damage-frequency relationship can be determined.

The data bank created by the Preprocessor Program contains basic and computed data for each structure or group of structures in the flood plain, along with data bank directories that permit rapid retrieval of these data. The data are organized by stream reaches, a convention often employed by planners when managing and analyzing data by conventional means. The extent of these reaches is defined for the Nonstructural Analysis Package programs by the user on the basis of study needs and available data; the only requirement is that a discharge-frequency function and an elevation-discharge function be provided at an index point on the stream in each reach. Hydrologic, hydraulic, and economic characteristics of the flood plain generally dictate the boundaries

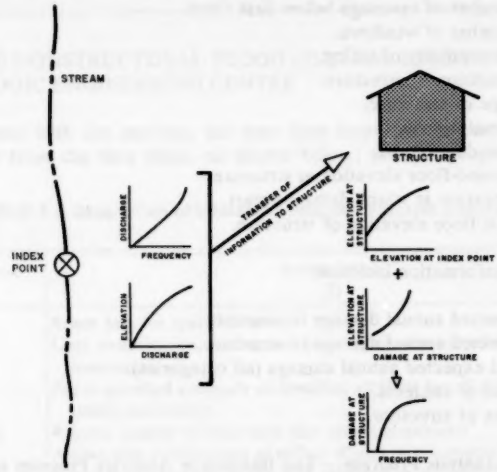

FIG. 1.—Schematic of Function Manipulation for Nonstructural Analysis

of stream reaches. For each structure in the flood plain, approx 150 elements of basic and computed information are stored in the data bank. This information includes 32 so-called structure attributes that characterize the structure and the nature of the flood problem at the structure. Structure attributes are grouped according to flood hazard, engineering, economic environmental, and historical information. These attributes, are used frequently for identifying and screening nonstructural measures.

Flood hazard information includes:

1. The depth of flooding for a 2-year event.
2. The depth of flooding for a 5-year event.
3. The depth of flooding for a 10-year event.
4. The depth of flooding for a 15-year event.

5. The depth of flooding for a 25-year event.
6. The depth of flooding for a 30-year event.
7. The depth of flooding for a 50-year event.
8. The depth of flooding for a 75-year event.
9. The depth of flooding for a 100-year event.
10. The recurrence interval of an event at first floor.
11. The level of protection.

Engineering information includes:

1. The construction material—frame.
2. Construction material—siding.
3. The number of openings at first floor.
4. The number of openings below first floor.
5. The number of windows.
6. The permeability of siding.
7. The condition of structure.
8. The type of structure.
9. The structure use.
10. The foundation type.
11. The ground-floor elevation at structure.
12. The elevation at which damages start.
13. The first-floor elevation of structure.

Economic information includes:

1. The expected annual damage to contents.
2. The expected annual damage to structure.
3. The total expected annual damage (all categories).
4. The value of contents.
5. The value of structure.

Interactive Analysis Program.—The Interactive Analysis Program is designed to retrieve on demand the basic and computed data for any structure or group of structures in the flood plain, to display the information in a convenient format for the water resources planner, and to manipulate the data and recompute the various flood hazard indices to demonstrate the efficiency of any proposed alternative-flood-control measure. Execution of the program is controlled by the user with commands that are similar to simple English and define the task to be accomplished and the structures affected. These commands are listed in Table 1. This problem-oriented language allows even the inexperienced user to move around in the program, going ahead when satisfied and reiterating when other alternatives are to be evaluated; the planner's judgement makes decisions, not the programmer's judgement.

The basic syntax of the commands for the Interactive Analysis Program is COMMAND CONSTRAINT, where COMMAND specifies the task and CONSTRAINT defines the affected structures. The constraints are defined in a similar way to the constraints of a mathematical programming problem: the left-hand-side of the constraint may be any of the 32 basic structure attributes, the operator

may be either LE (less than or equal), EQ (equal), or GE (greater than or equal), and the right-hand-side of the constraint may be alphabetic or numeric as appropriate for the specified attribute. Constraints may be combined with the operators AND and OR. Additional details are presented in Ref. 4.

Illustration of Interactive Program Use.—When execution of the Interactive Analysis Program begins, a banner identifying the program is printed, as shown below, and a title block, read from the user's data bank, is printed to avoid possible confusion of data banks.

```
**********************************************************************
*                                                                    *
*    PROGRAM FOR INTERACTIVE NONSTRUCTURAL ANALYSIS                  *
*                                                                    *
**********************************************************************

SANTA FE NONSTRUCTURAL FLOOD CONTROL STUDY
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To proceed with the analysis, the user then identifies the reaches that are to be read from the data bank, as shown below, and the program responds

TABLE 1.—Description of Interactive Analysis Program Commands

Command (1)	Action taken (2)
REACH	Reads data for specified reach
LIST	Lists identification number and name of all structures that satisfy constraints
PRINT	Prints specified summary or specified attribute for all structures that satisfy constraints
NUMBER	Reports number of structures that satisfy constraints
REVISE	Allows data modification to reflect effects of nonstructural alternatives
SAVE	Rewrites random access file with changes for nonstructural measures
TERMINATE	Terminates program execution

with a count of the structures in each reach or with an appropriate message if the requested reach is not in the data bank. User input in this and all examples that follow is in lower case.

```
reach 4 5
REACH          4 READ.          58 STRUCTURES.
REACH          5 READ.          18 STRUCTURES.

REACHES READ     2
```

The number of structures that satisfy specified conditions is determined using the NUMBER command. For example, to determine how many single-story structures without basements have an existing level of protection less than or

equal to 15-years, the user enters the command: NUMBER TYPE EQ 1SNB AND LEVEL LE 15. Here TYPE is the first attribute used in the screening, and 1SNB is the userdefined abbreviation for the desired structure type. LEVEL is the second attribute, LE indicates the comparison required, and 15 is the desired bound. In this case, the program responds: 2 FEASIBLE STRUCTURES.

To obtain a list of all structures that satisfy specified conditions, the LIST command is used. For example, the command: LIST TYPE EQ 1SNB AND LEVEL LE 15 yields identification numbers and names of all structures that satisfy the constraints, as follows:

11303. 1129 ALAMEDA (ALSO 11304)
11502. 1A PEQUENO (ALSO 11503)

Various data summaries can be produced using the PRINT command. For example, to display economic information for structure 11502, the user would enter the following command: PRINT ECONOMIC IDNO EQ 11502 where IDNO is the abbreviation for identification number. The program would produce the following summary:

ECONOMIC SUMMARY—STRUCTURE 11502.
1A PEQUENO (ALSO 11503)
VALUE STRUCTUR 40.00
VALUE CONTENTS 20.00

CATEGORY	CURNT EAD	BASE EAD
STRUCTUR	.79	1.07
CONTENTS	.68	.91
LAWN	.04	.05
TOTAL	1.50	2.02

The first and second lines of this summary identify the structure, and lines 3 and 4 give the estimated value, in thousands of dollars, of the structure and its contents. The table that follows shows the expected annual damages in three user-defined categories and the total for these values. If the user had simulated earlier some nonstructural flood-control measure for this structure, the column headed CURNT EAD would show the expected annual damages if the measure is implemented. The values in the column labeled BASE EAD are expected annual damages with no mitigation measures, and thus will not change.

Using the same PRINT command, a summary of flood hazard information may be obtained, as shown below.

HAZARD SUMMARY—STRUCTURE 11502.
1A PEQUENO (ALSO 11503)

REC INT	DEPTH
2.	0.00
5.	0.00
10.	0.00
15.	0.00
25.	.61

30.	.68
50.	.95
75.	1.54
100.	1.90

13. 1ST FLOOR
17. LEVEL OF PROT

PROTECTED	2.00
RAISED	0.00

This summary shows for events of various recurrence intervals (REC INT), the depths of flooding relative to the elevation at which damages start. The recurrence intervals of the events corresponding to the first-floor elevation of the structure and to the elevation at which damages start also are shown.

The engineering (structure-related) information may be displayed in a summary table. An example of this table is presented below. The ground elevation, first-floor elevation, and elevation at which damages start are shown, both for existing conditions (ORIG ELEV) and for modified conditions if a nonstructural measure has been specified. Additional structure attributes are displayed.

STRUCTURE SUMMARY—STRUCTURE 11502.
1A PEQUENO (ALSO 11503)
PHOTO ID 2.60

	CURNT ELEV	ORIG ELEV
GROUND	7131.00	7131.00
1ST FLOOR	7132.00	7132.00
START DMG	7133.00	7132.00
USE	RES	
TYPE	1SNB	
PERMEABLTY	HI	
	OPENINGS	
BELOW 1ST	0.	
AT 1ST	0.	
WINDOWS	0.	
FOUNDATION	SLAB	
FRAME MATL	ADOB	
SIDE MATL	STUC	
CONDITION	GOOD	

Tables of attributes for selected structures can also be generated using the PRINT command. For example, the command: PRINT EADT TYPE EQ 1SNB AND LEVEL LE 15 causes the program to identify all structures that satisfy the constraints and to print the total expected annual damages (abbreviated EADT) for those structures. In this case, the program responds

IDNO	EADT
11303.	1.32
11502.	2.02

Nonstructural flood control measures are "implemented" by using the REVISE command and by specifying the effects of the measure. For example, to evaluate the effects of providing nonstructural measures for the single-story structures with a level of protection less than or equal 15 yr, the user enters the command: REVISE TYPE EQ 1SNB AND LEVEL LE 15. When the user enters the appropriate information to indicate the types of and dimensions of the measures, data in memory for the structures is altered to reflect the modifications. The data bank is modified permanently only if the SAVE command is used.

Additional examples of program use and additional description of the commands are presented in Ref. 4.

Application of Nonstructural Analysis Package.—The computer programs of the Nonstructural Analysis Package have been used by the staff of the Hydrologic Engineering Center (HEC) in development of a nonstructural flood control plan for Santa Fe, New Mexico.

Setting.—The portion of the Santa Fe River flood plain studied is approximately 14.75 mi long, includes 721 acres, and is located primarily within the corporate limits of the city of Santa Fe. The location of the flood plain is shown in Fig. 2. The study area includes 457 structures within the limits of the 500-year flood plain; 80% of these are residential structures. Most of the structures are constructed of adobe, concrete block, or stucco-covered brick. Total capital investment within the flood plain is estimated at $64,000,000. Expected annual damages are estimated at $100,000.

Evaluation of Existing Hazard.—To evaluate the existing hazard, hydrologic, hydraulic, engineering, and economic information was integrated using the programs of the Nonstructural Analysis Package. The required discharge-frequency data was estimated via a regional frequency analysis performed by personnel of the Albuquerque District of the Corps of Engineers. That office provided the necessary discharge-depth data and depth-damage functions for the flood plain structures. Field surveys were conducted to obtain additional structure data, and all data were encoded for computer processing. The Preprocessor Program was executed, and a data bank was created and was stored on a commercial computer system. This data bank was then accessed with the Interactive Program to display selectively the indices of flood hazard for individual structures or for groups of structures.

Plan Formulation.—In this study, the Nonstructural Analysis Package proved most useful as a tool for data organization and display in the plan formulation step, both in the office and in the field. To formulate effective solutions to flooding problems, the planner must visit the site to develop an understanding of critical information that is not quantitative and thus cannot be captured in a data bank. However the quantitative information must be readily available for integration with this nonquantitative data. This integration is possible with the Interactive Analysis Program. Once the data bank is established and stored, it can be accessed with an interactive terminal via telephone. During this study, the data bank was accessed from the HEC's office in Davis, California, from

a Corps of Engineers office in Albuquerque, New Mexico, and from a motel room in Santa Fe. In the latter case, planners were able to identify structures for which nonstructural measures appeared to be economically and physically feasible and to inspect these structures on-site to verify this. (A field presence during formulation and evaluation is most important.) If the inspection indicated that the measure was not truly appropriate, the effects of an alternative measure could be easily analyzed. Often, this on-site inspection revealed errors in data and in subsequent calculations that lead initially to formulation of unacceptable

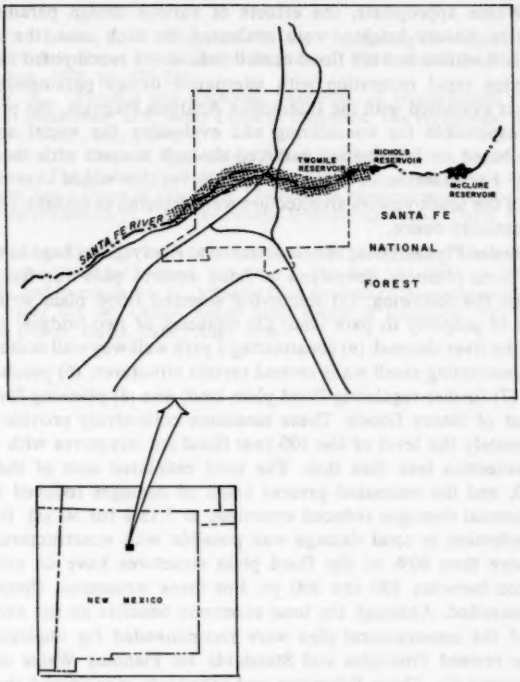

FIG. 2.—Location Map

flood-control plans. For example, errors were identified in data collected for and employed in development of the water surface elevation relationship for several structures. As a consequence of these errors, the expected damage reduction possible with nonstructural measures was miscalculated, and the initial estimates of design parameters were incorrect. Without the convenience of on-site inspection during the plan formulation, these errors might not have been discovered and corrected, or might have been discovered when the flood-control plan was presented for public inspection, thereby reducing the credibility of the entire study.

Plan Evaluation.—With the programs, the efficiency of nonstructural solutions to the flooding problem was assessed rapidly and easily, thereby allowing consideration of many alternatives. Simulation of implementation of most nonstructural flood-control measures is possible with the Interactive Analysis Program and is accomplished by identifying the measure and specifying design parameters. For this study, Corps engineers and planners, some of whom had little previous experience with computer utilization, evaluated a wide range of alternatives, including installing closures and other flood-proofing measures, constructing small levees, removing structures, and raising existing structures in-place. Where appropriate, the effects of various design parameters, such as alternative closure heights, were evaluated. In each case, the data in the data bank is modified and the flood hazard indices are recomputed immediately, thus, allowing rapid reiteration with alternative design parameters. As each alternative is evaluated with the Interactive Analysis Program, the program user must be responsible for considering and evaluating the social and political feasibility, based on information gathered through contact with those affected by flooding. For example, in Santa Fe, alternatives that would alter significantly the flavor of the southwestern architecture were rejected as socially unacceptable due to community desire.

Recommended Plan.—Using the Nonstructural Analysis Package as the primary analytical tool, planners developed a flood control plan for Santa Fe that recommends the following: (1) Removing selected flood plain structures and conversion of property to park land; (2) replacing of two bridges; (3) selected clearing of the river channel; (4) constructing a park walkway wall in the downtown area; (5) constructing small walls around certain structures; (6) purchasing flood insurance; (7) further regulating flood plain land; and (8) planning for assistance in the event of future floods. These measures collectively provide protection to approximately the level of the 100-year flood for structures with an existing level of protection less than that. The total estimated cost of the measures is $540,000, and the estimated present value of damages reduced is $266,000 (expected annual damages reduced amortized at 7.13% for 50 yr). In this case, no large reduction in total damage was possible with nonstructural measures because more than 80% of the flood plain structures have an existing level of protection between 100 and 500 yr. For those structures, flood insurance was recommended. Although the total economic benefits do not exceed costs, elements of the nonstructural plan were recommended for implementation in light of the revised Principles and Standards for Planning Water and Related Land Resources (5). These Principles and Standards recommend that plans be evaluated on the basis of combined economic (NED) and environmental (EQ) benefits, and thus implementation of a nonstructural plan for Santa Fe may be justifiable.

Summary

The Nonstructural Analysis package provides the water resources planner with the capability to manage, conveniently, the large quantity of data necessary for formulation of a viable nonstructural flood-control plan. The package consists of two programs: (1) The Preprocessor Program that digests hydraulic, hydrologic, economic, and engineering information for each structure or group of structures

in a flood plain and prepares a random access data bank; and (2) the Interactive Analysis Program that selectively accesses, displays and manipulates the data. These Programs are applicable in assessment of the existing flood hazard, in formulation of nonstructural flood-control measure to reduce the hazard and in evaluation of the effectiveness of the measures. The program package has been employed successfully in development of a proposed nonstructural flood-control plan for Santa Fe, New Mexico.

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ENERGY COSTS AND PORTLAND WATER SUPPLY SYSTEM[a]

By William M. Elliott,[1] M. ASCE and Randy P. Hawley,[2] A. M. ASCE

INTRODUCTION

This paper examines the municipal water supply system of Portland, Oregon and discusses the city's changing role of energy. The energy concerns of the times have led the city to adopt several new policies including conservation, the development of hydroelectric power at existing water supply dams, implementation of operating procedures, and modification or replacement of selected pump units. These actions by the city have resulted in substantial savings and will produce an additional power source for the metropolitan area.

The future of the Portland system is being significantly affected by energy. The city's Bureau of Water Works response to these changes are offered. Solutions are being implemented to address the future needs of Portland's water users. This paper may stimulate others to review their actions and offer their comments on further actions the City may take.

Energy Setting.—The Columbia River, which drains most of Washington, Oregon, and Idaho, is second only to the Mississippi River in its runoff. The Columbia River system produces more power than any other river. This system has provided the Pacific Northwest with low cost hydroelectric power.

The northwest is entering a new era in energy resources. The additional generating capacity required for future energy needs must be satisfied by other types of resources. Nuclear plants and thermal plants fed from the coal fields of Wyoming and Montana will supply the major portion of the future power needs. Power costs in the northwest are rising due to this increasing dependence on nuclear and thermal power sources. Over the last ten years, the cost of power has risen from 31 mills to 198 mills per kWh, an increase of 639%. The cost of electrical energy has become a significant portion of the Portland

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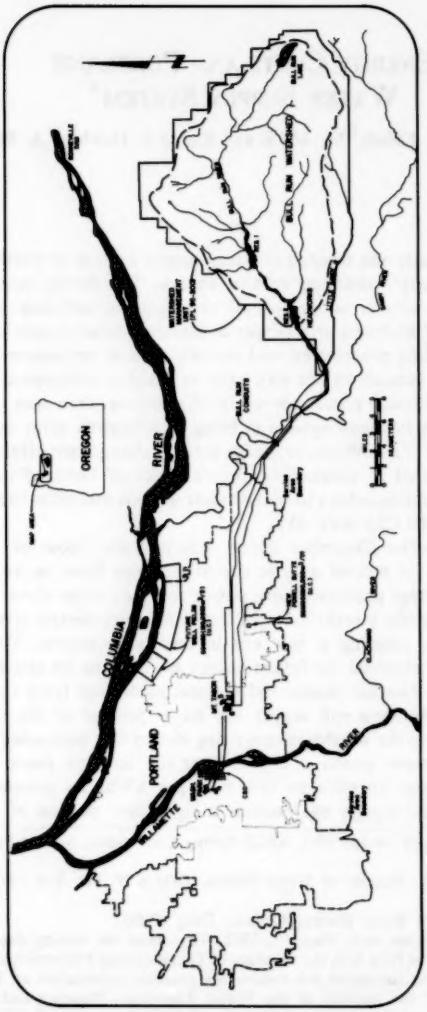

FIG. 1.—Portland Water Supply System

Water Bureau's operating budget. There is no relief in sight for rapidly escalating costs.

The Portland System.—The city of Portland, Oregon provides the municipal water supply to a 223 sq mile (579 km^2) area with a population of 700,000. The distribution area ranges in elevation from 10 ft (3 m) to 1,200 ft (366 m). A schematic of the system is shown in Fig. 1.

The city receives its water supply from the forested Bull Run Watershed located on the western slopes of the Cascade Mountains. The watershed is 102 sq miles (264 km^2) in area and is located within the Bull Run Watershed Management Unit. The Bull Run River is part of the Columbia River system. Last year, Congress reaffirmed its intent that this watershed serve the water supply needs of Portland. Public Law 95-200 was passed and signed by the President. This new law states, "The unit . . . shall be administered as a watershed. . .," and mandates management plans to assure the production of high quality water.

Water Storage and Diversions.—Diversions from the source started in 1894, with Bull Run Lake, a natural impoundment, providing the only major storage, 12,300 acre-ft (15,000,000 m^3). Supply to the city was simply a diversion of the river flow at the headworks into a pipeline. Increasing water demands required the construction of Dam No. 1, completed in 1929. Lift gates were added in 1954 to increase the available storage to 31,000 acre-ft (38,000,000 m^3). Dam No. 2 was completed in 1962, adding another 21,000 acre-ft (26,000,000 m^3) of storage to the system, to meet increasing water needs.

Three major pipelines, or conduits, each 25 mile (40 km) in length, supply water by gravity to terminal storage inside the city. This gravity delivery system has a capacity of 225,000,000 gal per day (852,000,000 L per day). One hundred thirty-six million gal (515,000,000 L) of terminal storage is provided at Mt. Tabor, where three large open reservoirs are located. Seven gravity supply mains from Mt. Tabor cross the Willamette River to serve the west side. Five of these mains feed two major storage reservoirs in Washington Park with a capacity of 34,500,000 gal (131,000,000 L). Water must be pumped to areas of the city with higher elevations than the Mt. Tabor and Washington Park reservoirs. Pumped water accounts for approximately 22% of the total water supplied by the city.

Factors of Change

A combination of events affecting long-range planning has offered Portland an opportunity for major long-term energy savings. The ingredients include an emergency source development program, terminal storage construction, and conduit interties and contracts to gain long-term service commitments with outside users. These users are water districts and cities who in turn supply portions of the metropolitan area outside the city. Portland will alter the hydraulic gradient of the primary transmission system to meet future water needs. This change in operation will result in a significant reduction in the use of energy for pumping.

New Terminal Storage.—Additional storage was needed to provide a terminal point for the city's emergency water source—well fields adjacent to the Portland International Airport. This terminal storage, under construction on Powell Butte in southeast Portland, will also receive water from the existing and future conduits.

This 50,000,000 gal (189,000,000 L) storage facility will be the third largest buried reservoir on the west coast. To coordinate the operation of the new Powell Butte Reservoir with the ground-water supply and the supply conduits, conduit interconnections were needed. This system tied all supply components together.

The elevation of the Powell Butte storage was determined by the hydraulics of the conduits, the integration of the well supply into the existing system, service pressures, and topography of the reservoir site. The elevation chosen to best fit the needs of Portland also changed the hydraulic grade line to the terminal storage point. The flexibility of the system allowed a change in the hydraulic grade line with a minimal loss in conduit capacity. This flatter energy gradient, coupled with a large diameter pipeline, is sufficient to deliver water by gravity over Portland's West Hills to eastern Washington County. This area is currently supplied by pumping. Fig. 2 displays the existing and proposed hydraulic grade lines.

Increasing Suburban Demand—Gravity Supply.—Eastern Washington County, west of Portland, has experienced steady growth in population, commerce, and industry. The majority of the east Washington County area relies on Portland's system for its present and future water supplies. The growth in water demand is expected to continue. To meet the long-term water supply need, Portland proposed the construction of a large capacity supply conduit. The pipeline would extend from the Powell Butte Reservoir to the Washington County line. Total length of the main is 14 mile (22.5 km). Fig. 3 displays the 60 in. (1,524 mm) pipeline that will be used to supply Washington County needs into the next century.

The major reasons for selection of the gravity supply conduit over other means of meeting the water needs of eastern Washington County were energy savings and energy costs. These reduced costs are noted (2).

> The project will save money by conserving energy. For the first time, gravity water will flow the entire distance from the Bull Run watershed to eastern Washington County. The gravity supply main will reduce pumping requirements by 50 percent and the increasing energy costs associated with it.

Before this supply project could move forward, agreement on new long-term contracts with the city's outside purveyors (water districts and cities) was required. The completion of these agreements will provide Portland with the basis to make the needed capital investments.

Grade Line Implications for Energy Savings.—The construction of an all-gravity supply to the city's suburban customers is a breakthrough that will reduce energy needs for the future and provide gravity service to nearly all customers. Certain areas of the city and of higher elevation in Washington County will still require pumping, but the need to expand pumping capacity to meet increasing demand will be nullified.

Electrical Energy Rates and Costs.—The rapid escalation in energy rates for the pumping of water is shown in Fig. 4. Energy costs have increased over 600% in the last ten years. The proposed gravity supply will dramatically reduce future energy needs. Fig. 5 compares the projected annual energy costs to

deliver water to Washington County users with and without the gravity supply in service. At current demand levels, twelve million kWh and $250,000 annually would be saved.

Hydroelectric Development.—The opportunity to add turbines and generators at the two major dams in the watershed has been a long sought-after dream. Penstocks were included in the original construction of the dams. The chance

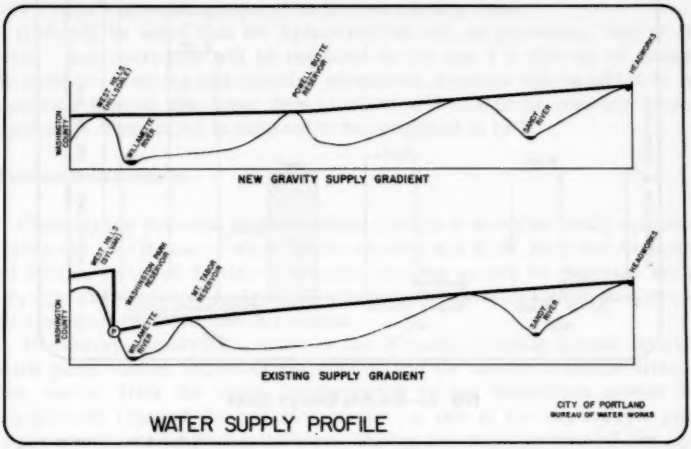

FIG. 2.—Hydraulic Gradient to West Portland

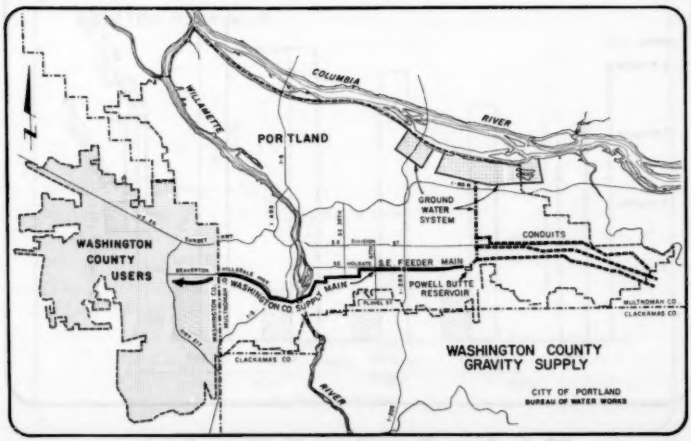

FIG. 3.—Gravity Supply Line to Washington County

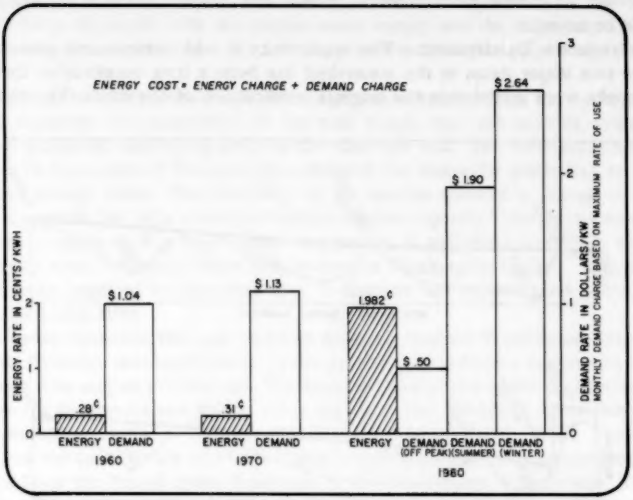

FIG. 4.—Electric Energy Rates

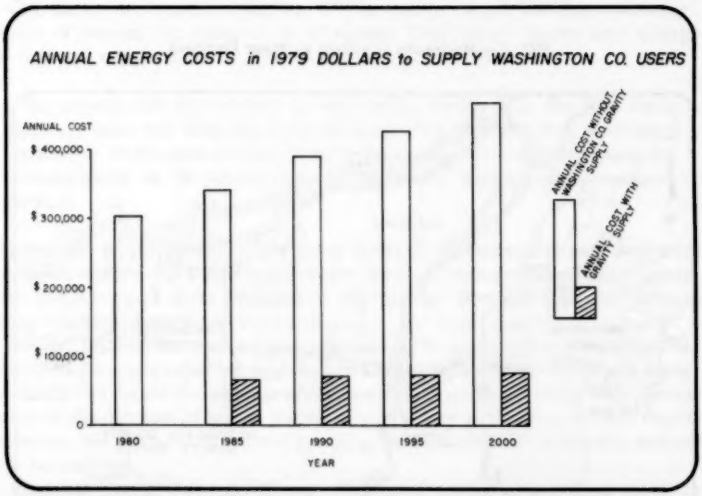

FIG. 5.—Energy Cost to Supply Washington County

to move ahead started in 1977 when the Commissioner of Public Utilities initiated a review of the financial and engineering feasibility of the project.

A project was developed to use the existing water supply dams to generate electrical power. The Portland City Council seized the opportunity by authorizing a project to move forward with construction of the generating facilities. The city is currently taking bids for powerhouse construction, and have on order the generating machinery. This $45,000,000 project will add a 24 megawatt (MW) and a 12 MW generating capacity at the two existing dams.

It should be noted that the hydro-facilities will be generating "run of the river," and operations will be restricted to the top 2 ft (0.6 m) of storage. In addition, a mixing and selective withdrawal structure will be added to the upstream face of the upper dam to allow withdrawals to replicate current operations. The project is expected to be completed in 1982.

Pumping Enhancements

Changing the hydraulic gradient relates directly to an earlier study completed by the city. The Bureau of Water Works, working with R. W. Beck and Associates of Seattle, reviewed Portland's westside pumping system for potential energy savings. This study was made possible by a previous project for the development of a computer network analysis system.

The energy conservation study for the Westside Pumping System reviewed each pump station, individual pump units, and the service condition affecting the station. Over the years, improvements to the distribution system had significantly changed the hydraulic gradient at two of the city's major pump stations since its original installation. During the major portion of the year, nonpeak, the pumps were operating below optimum efficiency. Two approaches

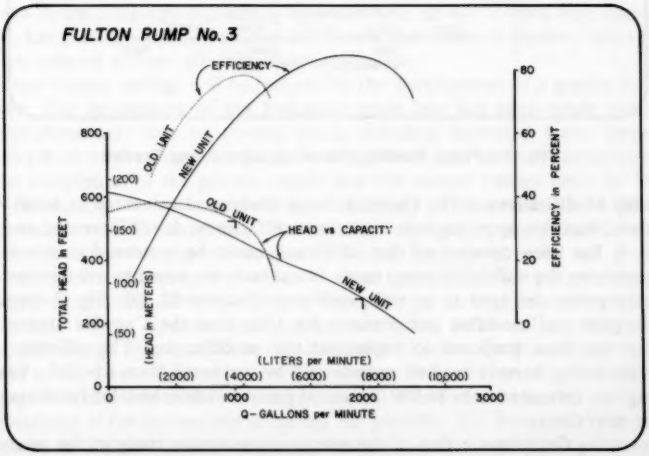

FIG. 6.—Pump Replacement at Fulton Pump Station

were taken to improve pump station efficiency and reduce energy costs: (1) Pump replacement; and (2) pump modification.

Pump Replacement.—One of the pumps at Fulton Pump Station, in southwest Portland, was found to be operating at approximately 65% efficiency, during normal demand periods, due to increased suction head. After an evaluation of each unit in the station, two pumps were scheduled for replacement. Now, the efficiency of the new pump units is nearly 85% during nonpeak demands, reducing energy requirements. Fig. 6, following, shows the pump curves for pump No. 3 at the Fulton station.

The energy savings alone will offset the cost of replacement in approximately 8 yr. The new pumps move about twice as much water during normal demand periods. During peak demand periods, the pump units deliver nearly the same quantity as the units that were replaced with a sacrifice in efficiency.

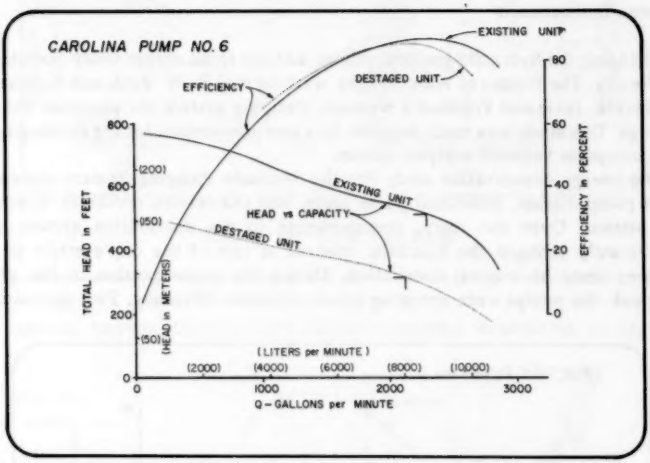

FIG. 7.—Pump Modification at Carolina Pump Station

Pump Modifications.—The Carolina Pump Station, also located in southwest Portland, was also operating below maximum efficiency, due to increased suction head. It has been determined that efficiency could be increased substantially by modifying the individual pump units. In one unit, the impeller will be trimmed and the pump destaged at an estimated cost of about $2,500. Fig. 7 displays the original and modified pump curve for Unit 6 at the Carolina station. A project has been proposed to implement this modification. The efficiency of the unit during normal demand periods will be increased from 65–83%. Yearly savings are estimated to be $6,000. A second pump will be scheduled for destaging in the near future.

Operating Guidelines.—Part of the energy conservation study of the westside pumping system was an effort to develop some basic operating guidelines. An operating guide was developed for each of the major stations which outlines

a general operation plan. The plan identified the following points:

1. Most efficient pump units.
2. Preferred starting sequence.
3. Primary pump units and peaking units for normal operation.
4. Primary pump units and peaking units for summer operation.
5. Rated flow of each unit.
6. Typical flow of each unit.

The pumps were ranked or listed by efficiency and delivery costs of pumped water. The demand charge for each unit was also tabulated.

The development of the operating guide has opened up valuable dialogue between the operators and the Engineering Division. The guidelines have also been used as an educational tool. Portland operators have become acutely aware of the rising costs of energy and have taken basic steps to reduce energy use. After the implementation of a variable demand rate schedule by Portland General Electric, the operators took immediate steps to minimize peak hour pumping. This has resulted in direct cost savings.

Ongoing Program.—The bureau has budgeted ongoing projects in its capital improvement program for pump modification and replacement. These programs will ensure further energy savings in the future as the various units are addressed.

Conclusions

The high level of public awareness of energy management dictates that all major energy consumers must look again to the basics. A review of how the Bureau operates energy-consuming systems, with an eye toward wise management, has paid dividends. Portland has shown that costs of existing operations can be reduced without affecting water deliveries.

Future energy savings will be realized by the development of a gravity supply source. The adjustment of the hydraulic grade line has been made possible by simultaneously meeting several needs including increased water demand, development of an emergency ground-water source, and reservoir construction.

The completion of the gravity supply line will reduce energy costs by 50%. The bureau expects an annual savings of energy on the order of 12,000,000 kWh based on 1979 water use. This savings represents nearly 7,100 barrels of oil. The city's emergency source of supply from deep wells will provide needed system reliability, but will add about 1,000,000 kWh per year to our energy requirements. This 1,000,000 kWh represents about 600 barrels of oil.

The start of the hydroelectric project at existing dams will provide a new source of energy long ignored due to cheap power costs. Further pump replacements to improve efficiency are still needed to fully realize all of the energy savings identified. In addition, the bureau will investigate improved operations to determine if further savings in energy are possible. The bureau has budgeted programs to address these on a priority basis.

Basic thinking concerning how electrical energy is used offers many new opportunities to save and conserve . . . if only the time is taken to look.

Appendix I.—References

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ACCESS TO WATER-RELATED DATA IN TEXAS[a]

By David L. Ferguson[1]

INTRODUCTION

The Texas Natural Resources Information System (TNRIS) is a mechanism within the Texas state government for storing, processing, and disseminating data and information relating to the state's water and other natural resources. It represents a cooperative effort among thirteen of the state's natural resource-related agencies and is governed by a task force composed of one representative from each agency. The objective of the TNRIS is to make natural resource and environmental data and information more easily accessible so that those with responsibilities in the natural resouces fields can carry out their tasks more thoroughly and efficiently.

This presentation will give a brief history of the development of the TNRIS, an overview of its organizational framework, an indication of the types of water-related data contained in the system, and an explanation of how the system functions within the Texas state government.

BACKGROUND AND DEVELOPMENT

The TNRIS had its origins in the mid 1960's with the establishment by the Texas state legislature of the Texas Water Oriented Data Bank. Eight state agencies participated in this early effort. Initial activities of the data bank included the cataloging of water-related data in concert with the work of the U.S. Geological Survey's Office of Water Data Coordination. These efforts resulted in the first comprehensive inventory of water-oriented data in Texas state agencies.

In addition to assisting users in locating hydrologic information, the data bank provided a basis for eliminating duplication in data procurement. It was found, for example, that several state agencies were purchasing National Weather Service information on a recurring basis. Once identified, areas of duplication such as this were easily eliminated.

[a]Presented at the April 14–18, 1980, ASCE Convention & Exposition and Continuing Education Program, held at Portland, Oreg. (Preprint 80-162).

[1]Secretary, Texas Natural Resources Information System Task Force, and Chf. of Information Systems and Services, Tex. Dept. of Water Resources, Austin, Tex.

Note.—Discussion open until March 1, 1982. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on October 9, 1981. This paper is part of the Journal of the Water Resources Planning and Management Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. WR2, October, 1981. ISSN 0145-0743/81/0002-0375/$01.00.

By the early 1970's it was realized that a broader, more comprehensive information system was needed and in 1972 work was initiated to establish the Texas Natural Resources Information System. State agency participation was expanded to include fifteen state agencies and the TNRIS Task Force was established with representation from each to guide the development and operation of the system. By 1980, after a merger of the three state water agencies, the number of participating agencies was reduced to thirteen. Several other agencies and offices participate in an *ex officio* status.

List of TNRIS Participants

Texas Department of Water Resources
General Land Office
Texas Forest Service
Texas Industrial Commission
Texas Department of Health
Texas Air Control Board
Texas Coastal and Marine Council
Texas Department of Agriculture
Railroad Commission of Texas
Bureau of Economic Geology, University of Texas at Austin
State Department of Highways & Public Transportation
Texas Parks & Wildlife Department
Texas State Soil & Water Conservation Board

Ex officio members:
Texas Historical Commission
Texas Department of Community Affairs
Governor's Office

Organization and Operation

TNRIS exists primarily to serve the participating agencies by helping them carry out their legislative mandates. The system encourages coordination among member agencies by participating in joint projects aimed at developing capabilities for natural resource management. This constitutes one of the system's significant strengths. TNRIS has not been conceived to take away agency control, nor to establish a massive data bank. Instead of attempting to centralize all natural resource data, TNRIS ties together the information systems already existing in the state and elsewhere, in order to make the data more easily accessible.

The organizational concept of TNRIS (Fig. 1) may be described as: (1) A linked network of user entities acquiring and maintaining natural resouces data; (2) a staff which provides a focal point for information on data availability, procurement, and analysis; and (3) a central facility with computer capability to handle storage, retrieval, processing, analysis, and, where appropriate, presentation of natural resource data and information.

The TNRIS offices and staff are housed within the Texas Department of Water Resouces. The staff consists of systems analysts, computer programmers, remote sensing specialists, and user support personnel.

The task force elects a chairman and vice-chairman biennially. The task force secretary is designated by the executive director of the Texas Department of Water Resouces from the department's program area which provides support to TNRIS. The estimated expenditures relating to TNRIS establishment and

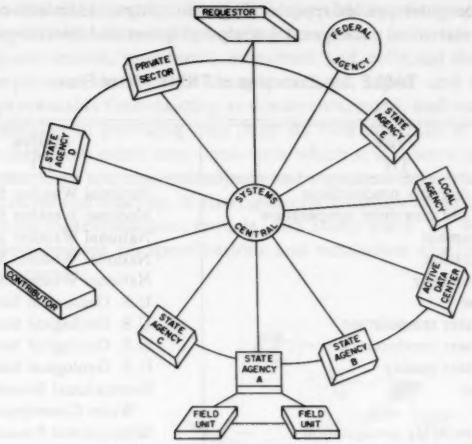

FIG. 1.—TNRIS Organizational Concept

operation from 1969 through 1979 total $2,469,000. Of this total figure, personnel costs of participating agencies are estimated to be $520,000. Costs for personnel have aggregated some $1,111,000. Other operating costs, including computer time, data, and supplies have run to about $838,000.

Data in System

Data in the TNRIS is grouped into the following six categories: (1) Hydrological; (2) meteorological; (3) geological; (4) biological; (5) socioeconomic; and (6) base data. The latter category includes maps and remotely sensed data. Currently more than 300 automated files and over 150 manual files are indexed in the system (Table 1). Numerous other data sources are routinely queried by computer terminal, telephone, or letter in response to user requests (Fig. 2).

Bibliographic data are available to TNRIS users through two online private sector sources: (1) The System Development Corporation's ORBIT system; and (2) the Lockheed Corporation's DIALOG system. Access to data and information from certain of the federal systems is accomplished through private sector contractors.

From the wide variety of data in the system it is clear that the TNRIS has not limited its files to natural resource materials in the strictest definition of the term. This is due to the fact that most natural resource projects or studies must take into account a wide range of information including socioeconomic and other types as well. The multidisciplinary approach is an important aspect of the TNRIS philosophy.

Capabilities and Services

TNRIS operational capabilities and services are varied and extensive. They

include: (1) Computer-printed reports; (2) graphic output; (3) interface with remote terminals; (4) statistical packages; (5) computer-generated microform; (6) geocod-

TABLE 1.—Examples of TNRIS Water Files

Type (1)	Source (2)
Daily and monthly precipitation	National Weather Service
Minimum and maximum temperature	National Weather Service
Wind movement	National Weather Service
Pan evaporation	National Weather Service
Relative humidity	National Weather Service
Streamflow	U.S. Geological Survey
Surface water temperature	U.S. Geological Survey
Surface water conductance	U.S. Geological Survey
Surface water quality	U.S. Geological Survey
Streamflow	International Boundary and Water Commission
Daily and monthly precipitation	International Boundary and Water Commission
Gross and net lake surface evaporation	Texas State Agencies
Suspended sediment	Texas State Agencies
Reservoir content	Texas State Agencies
Coastal zone hydrographic data	Texas State Agencies
Ground water quality	Texas State Agencies
Water well measurements	Texas State Agencies

TABLE 2.—TNRIS Interface with Federal Systems

Federal systems (1)	TNRIS interface (2)
USGS (United States Geological Survey)	NAWDEX (National Water Data Exchange)
	WATSTORE (Water Data Storage and Retrieval System)
	NCIC/EDC (National Cartographic Information Center, EROS Data Center)
EPA (Environmental Protection Agency)	STORET (Storage and Retrieval System: Water Quality Data)
Department of Commerce	NTIS (National Technical Information Service)
	Census (1970 U.S. Population census)
NOAA (National Oceanic and Atmospheric Administration)	NWS (National Weather Service)
Department of Agriculture	FAPRS (Federal Assistance Programs Retrieval System)
Department of Energy	RECON (Remote Console: For accessing selected water resources abstracts)

ing/geographic information handling; (7) analysis of remotely sensed data; (8) catalogs/indexes; (9) responses to inquiries concerning the availability of compu-

terized data, aerial photography, satellite imagery/data, cartographic products, and technical publications; and (10) ordering services.

TNRIS indexes sensed, monitored, measured, and collected data existing in both machine-processable form as computer cards, tapes, and disks, and in nonmachine-processable form existing as documents, maps, and remotely sensed imagery. In addition to providing data from its own information base, TNRIS disseminates data from other data basis with which it is interconnected, refers inquiries to other data sources, and adjusts and organizes data into forms suited to storage/retrieval and analysis. It also functions in manipulating and processing data into graphic representations, models, and study plans. Its work may also lead to the development of specifications and simulation systems for natural resources management.

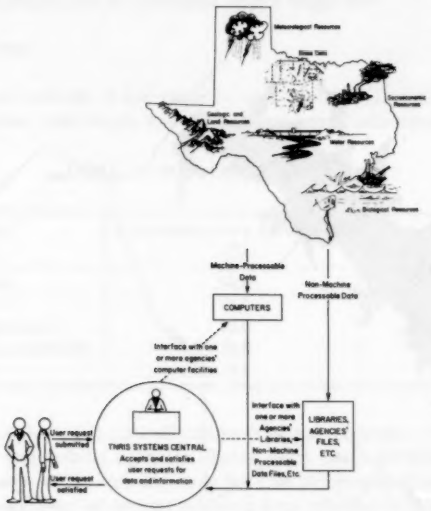
Meteorological Resources
Mineral Resources
Geologic and Land Resources
Water Resources
Biological Resources
Machine-Processable Data
Non-Machine Processable Data
COMPUTERS
Interface with one or more agencies' computer facilities
User request submitted
User request satisfied
TNRIS SYSTEMS CENTRAL
Accepts and satisfies user requests for data and information
Interface with one or more Agencies' Libraries, Non-Machine Processable Data Files, Etc.
LIBRARIES, AGENCIES' FILES, ETC.

FIG. 2.—TNRIS Data Storage and Retrieval Concept

TNRIS computing facilities include a Univac 1100/41 processor and a computer graphics system built around an Interdata 7/32 mini computer. Peripherals associated with the computer graphics installation include a Calcomp 7X* flatbed plotter, a Bendix digi-grid digitizer, a Ramtek color display, and a Matrix camera system. In addition, a variety of imagery handling devices of remotely sensed data are available to TNRIS users.

An ongoing educational program includes short courses on methods of accessing the TNRIS data files and on other aspects of the system. TNRIS regularly publishes several items including a newsletter which has a wide distribution and serves to make known the services which are available to users.

TNRIS remote sensing and cartographic activities can be classified into four

areas: (1) Indexing and cataloging; (2) data retrieval; (3) education and consultation; and (4) data analysis. As a state level affiliate of the National Cartographic Information Center (NCIC), TNRIS has been engaged in an extensive indexing effort involving all known sources of imagery for the state, including federal and state agencies, universities, and the private sector.

In its data retrieval activity in the remote sensing and cartographic area, TNRIS assists users in procuring imagery and map data. The TNRIS computer terminal interface with the EROS Data Center and the 16 mm and microfiche browse file of the data center's principal holdings are particularly helpful. This

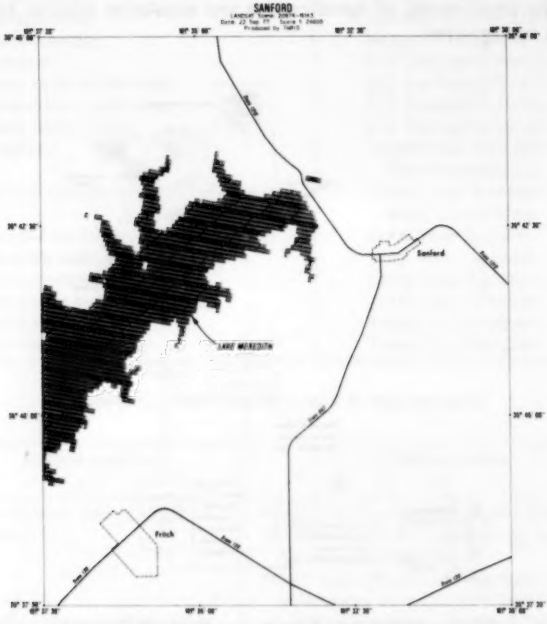

FIG. 3.—Sample Dam Inventory Plot Showing Water for Area in Texas Panhandle

equipment makes several hundred thousand frames of imagery covering the state of Texas available to TNRIS users.

In the education and consultation field, TNRIS has offered several short courses in image interpretation and remote sensing for personnel from governmental entities and the private sector. A four-part series given throughout the year includes "Fundamentals of Remote Sensing," "Air Photo Interpretation," "Landsat Image Interpretation," and Principles of Landsat Digital Processing."

In the data analysis area of its remote sensing and cartographic endeavors, TNRIS has been involved in a number of activities using Landsat satellite data to assist Texas state agencies in natural resource-related projects. One such

project led to a production use of Landsat digital data for updating the dam inventory maintained by the Texas Department of Water Resources (Fig. 3).

As one of several TNRIS cartographic data activities, the staff has been involved for several years in a project to develop a generalized Geographic Information System for storing and reproducing map-related data. The system manipulates data in topological (polygonal) form as well as grid cell form.

Using the Geographic Information System, data can be extracted from cartographic products in the form of areas, lines, and points; they can be stored on computer files, along with associated textual information, and can be reproduced in the form of map overlays. Base information such as soil type locations, biologic assemblages, oil and gas wells, pipelines, highway locations, and dam locations have been stored in the system. Using the TNRIS Geographic Information System, map data extracted from several different base maps at any scale or projection can be combined onto a single plot.

How System is Used

As TNRIS has evolved, it has come to serve quite a large user community, including industries, individuals and private businesses, educational institutions,

TABLE 3.—Use of TNRIS Data Files

Users (1)	Accessions, as a percentage (2)	Users, as a percentage (3)
State government	56.4	12.6
Industry	18.0	39.0
Educational institutions	10.0	14.2
Other government agencies	10.8	18.1
Others	4.8	16.1

municipalities, county governments, councils of governments, river authorities, water districts, and federal agencies (Table 3). A basic TNRIS philosophy is service to the users, whomever they may be. This philosophy extends to assisting the users in procurement of natural resource data, whether it is readily available through computer linkages, or whether it must be researched and located from inventories and catalogs of material located in a number of different places.

As TNRIS has developed, its staff has continually made an effort to provide users with quick turnaround on data requests, whether received by letter, telephone, or personal visit to TNRIS Systems Central. Depending on the manner in which a request is made and the source of the data, it may be satisfied that same day or it may require several working days.

In order to facilitate service to users who require direct and immediate access to data, the TNRIS Monitor has been developed. The monitor is a system by which users can access the data base through remote terminals in different parts of the state (Fig. 4). The monitor is designed for easy operation by nontechnical users. It provides ready access to over 75% of the more than 300 TNRIS automated natural resource and related data and information files. Files accessible through the monitor include those held by the TNRIS Systems

Central facility and those available from other computers in the immediate vicinity of TNRIS allowing linkage to users through TNRIS computer facilities.

TNRIS Monitor users receive top computer priority to ensure fast turnaround. Previous experience at using computer terminals is not necessary in order to utilize the monitor, since specific instructions are provided to the user during access. During the fiscal year ending August 31, 1979, 772 users made 2,546 different data requests generating 3,494 accessions to TNRIS files. These accessions produced a total of 4,088 sets of output products. Eighty-nine percent of the TNRIS data requests for the period were made either by telephone, letter, or in person. The remainder were via computer terminal. The average

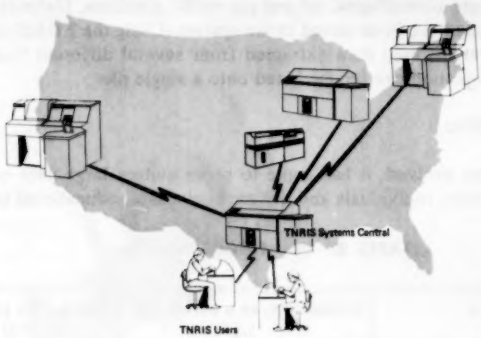

FIG. 4.—Online Access to TNRIS File and to Other Data Bases through TNRIS Monitor

processing time for requests initiated by letter, telephone or in person was three and one half days. Computer terminal accessions were processed on demand.

Conclusions

Over the years the TNRIS has been effective in making information more readily available, avoiding duplication of effort in data acquisition, and fostering cooperation among Texas state agencies. The success of the system may be attributable to a number of factors. Most important among them is, perhaps, the obvious need for and benefits of such a system. The most significant benefits from such a system include: (1) In most cases, users can be linked to the natural resource data they require with a single inquiry to TNRIS; and (2) TNRIS member agency staff time required to respond to user inquiries is substantially reduced. Other factors which have contributed to the success of TNRIS are: (1) Focusing on the development of the ability to assist its members in responding to their legislative requirements; (2) linking existing systems rather than centralizing all data holdings; and (3) provision by member agencies of direction for TNRIS through the TNRIS Task Force.

It is also noteworthy that TNRIS has evolved somewhat slowly rather than being created "full blown" over a short period of time. It is also important that instead of having to purchase expensive computer equipment, the system makes use of existing facilities. These and other factors might be taken into

account by other states in the process of planning for the development of natural resource information systems.

In response to numerous requests from users and as part of the educational program, there is considerable documentation available on TNRIS. Inquiries should be made to TNRIS Systems Central, P.O. Box 13087, Austin, Texas 78711.

COMPARISON OF DETENTION BASIN PLANNING AND DESIGN MODELS

By John R. Donahue,[1] Richard H. McCuen,[2] A. M. ASCE, and Timothy R. Bondelid[3]

INTRODUCTION

It is recognized that urbanization is often responsible for increases in both runoff flow rates and sediment volumes. In order to evaluate the efficiency of stormwater management (SWM) in controlling the effects of urbanization, it may be of interest to superficially examine the urbanization process. Urban development increases the percent of the drainage area that is covered by impervious surfaces and changes the drainage pattern such that the flow lengths are decreased. These physical changes have the effect of decreasing the natural storage of the catchment, which also causes changes in the timing characteristics of the runoff. The decrease in natural storage is evident from the increased peak rates, decreased times-to-peak, and increased volumes of runoff. Increased runoff erodes channels and ultimately deposits sediments in downstream channels.

To mitigate these detrimental effects, stormwater management policies have been adopted with the intent of limiting peak flow rates and sediment volumes from developed areas to that which occurred prior to development. While other SWM methods, both structural and nonstructural, are available, SWM basins appear to be the most popular. The SWM policies specify the intent of stormwater management. However, in many cases, the procedure by which the intent is met is left, at least in part, to the discretion of the planner and designer. This has led to the development and use of a multitude of methods in the planning and design of SWM facilities. These methods provide widely different estimates of the volume of on-site storage that is required to meet the intent of stormwater management. While the true volume of man-made storage that is necessary to replace the natural storage lost by development is not readily determined,

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the fact that different methods give significantly different estimates should be of concern to those involved in policy formulation and enforcement. It is also not unreasonable to assume that some of these design methods are not providing designs that meet the intent of stormwater management policies.

There is little hope that in the near future one method will be clearly identified as being the most accurate using either an empirical analysis or a theoretical development. Measured data are quite limited and not sufficient to identify a single model as being the most accurate in establishing the required volume of storage. Using data from a 6-month data collection program, Davis (5) and Davis, et al., (6), indicate that detention basins may have sediment trap efficiencies greater than 90% and control peak flow rates for smaller storm events to that occurring prior to development. Amandes and Bedient (2) analyzed data for a 200-acre watershed in Houston, Texas, and concluded that detention storage can provide significant water quantity and quality benefits in a developing watershed. While studies such as these are very useful towards advancing the state-of-the-art, they are an inadequate data source for comparing design methods and selecting conclusively the most accurate planning and design methods. Conceptual models based on theoretical concepts (Bondelid and McCuen, 4) have limitations because the available data are not adequate to test the degree to which simplifying assumptions reduce their accuracy.

Because a variety of methods have been proposed for sizing stormwater management basins, it is worthwhile to attempt to classify them according to dominant characteristics. Some methods include only components that represent the hydrology of the watershed while others include hydraulic routing elements of the detention facility. Methods having the former property are only useful for estimating the volume of required storage while those that include routing elements can provide a simultaneous evaluation of both the volume of storage and the sizing characteristics of the outlet facilities. For this study, methods that include both of these characteristics simultaneously will be referred to as design methods while those that only provide for computing a volume of storage will be referred to as planning methods. This separation is not unreasonable because the effectiveness of a stormwater management basin depends on the degree to which determination of the size characteristics of the outlet facilities are coordinated with the estimation of the storage volume.

Because stormwater management basins are considered a primary means of meeting the intent of SWM policies, it is useful to compare planning and design methods. Seven methods that are representative of those currently in use were selected. Each method was evaluated for a cross section of conditions and the resulting designs were compared.

Stormwater Management Planning and Design Methods

Because data for calibration are rarely available at a site, only methods that do not require calibration were selected. The following two methods were considered as design methods: (1) Baker (3); and (2) Bondelid and McCuen (4). The following four methods were considered as planning methods: (1) ILLUDAS (Terstriep and Stall, 11); (2) Abt and Grigg (1); (3) Wycoff and Singh (12); and (4) TR-55 (Soil Conservation Service, 9). The computed volume

of the natural storage lost due to development was also considered as the fifth planning method. A review of the important assumptions that form the basis of each method and that effect the sizing of detention facilities is provided.

Baker Method.—The Baker method is typical of most methods in that it is based on the premise that the volume of storage, V_s, equals the area between the inflow and outflow hydrographs of the direct runoff after development. Recognizing that the rate of outflow from the detention basins is dependent upon the depth of impounded water, the Baker method uses the linear routing equation to route flow through the detention basin. Peak rates of runoff are determined using the rational method, and trapezoidal hydrographs are used to represent the time distribution of runoff. The solution procedure is iterative in that rainfalls of various durations are used to determine the critical rainfall duration (i.e., the duration that requires the largest storage volume).

Bondelid Method.—The Bondelid method is a computerized optimization procedure for design of stormwater management basins. The method considers the relationship between inflow, outlet structure configuration, basin storage characteristics reservoir routing, and outflow. The before development and after development runoff hydrographs are computed using the TR-20 computer program (SCS, 10). The program designs basins and outlet structures so that flows of two return periods are controlled simultaneously. The outlet structure consists of a riser pipe with a circular orifice at the base of the riser pipe. The orifice is used to control the smaller flow and the riser is used to control the larger flow. Basin storage characteristics are determined by considering rectangular basins with variable side slopes. For given orifice and riser diameters, the basin side slopes are adjusted so that the outflow peak discharges for the flows of different return periods are approximately equal to the before development peak discharges of the same return periods. The method utilizes the reservoir routing procedure contained in TR-20. The required volume of storage is determined from the peak water surface elevation. A large number of combinations of orifice and riser diameters are considered in each computer run and specified with the output. The orifice and riser combination that requires the least storage, but still meets the SWM policy criteria, can, therefore, be determined.

ILLUDAS Method.—The Illinois Urban Drainage Area Simulator, ILLUDAS, provides a method either for the design of new drainage systems or for the evaluation of existing systems in urban areas (Terstriep and Stall, 11). The ILLUDAS provides several options for determining the size of stormwater detention basins. For the case of evaluating an existing drainage system, ILLUDAS will accumulate all design storm flow exceeding the maximum capacity of the conveyance structure for each reach. The maximum required detention storage, which equals the accumulated volume, is then known for each subbasin. For designing a new drainage system the options include specifying the maximum volume of available storage, the maximum discharge through the reach, or the desired outlet pipe size. For a given subbasin, either an allowable detention volume is specified or the volume of storage is controlled by pipe size or maximum discharge. The ILLUDAS does not allow for both constraints on a particular reach. When the volume of available storage is specified, ILLUDAS will allow incoming flow to fill the allowable storage while providing an outlet capacity that will make effective use of this storage. In this study, the maximum allowable discharge through the reach is set equal to the subbasin predevelopment peak.

The volume of detention storage is equal to the peak cumulative volume of flow exceeding the specified peak.

Abt and Grigg Method.—Abt and Grigg (1) provided "an approximate method for sizing detention reservoirs." The volume of storage equals the area between the inflow and outflow hydrographs. The rising limbs of the inflow and outflow hydrographs are assumed to coincide until the limiting discharge rate (i.e., the before development peak discharge) is reached. The outflow and inflow hydrographs are assumed to be trapezoidal and triangular, respectively. For a single detention basin the volume of storage, V_s, is given by:

$$V_s = \left(\frac{1+m}{2}\right) Q_p t_c (1-\alpha)^2 \left(\frac{60}{43{,}560}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which m = the ratio of the duration of the hydrograph recession to the time-to-peak; t_c = the time-of-concentration; Q_p = the peak discharge; and α = the ratio of the peak discharges for the outflow and inflow. The method assumes that the time-to-peak equals the time-of-concentration and that the peak rate of outflow from the detention basin equals the peak discharge that occurred prior to development. Methods for computing the peak discharge, volumes of runoff, and times of concentration were not recommended.

Wycoff and Singh Method.—Wycoff and Singh (12) provide a method for making "preliminary hydrologic designs of small flood detention reservoirs." The following relationship for computing the volume of storage was evaluated using a regression of data generated from a more detailed hydrologic model:

$$V_s = \frac{1.291\, V_r \left(1 - \dfrac{Q_0}{Q_i}\right)^{0.753}}{\left(\dfrac{T_b}{T_p}\right)^{0.411}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which V_r = the volume of runoff; Q_0 and Q_i = the peak discharge rates of the outflow and inflow, respectively; and T_b and T_p = the time base and the time-to-peak of the inflow hydrograph, respectively. The time base is measured from the start of runoff to the time when the discharge on the recession equals 5% of the peak discharge rate Q_i. For calibration of Eq. 2, values for the ratio Q_0/Q_i varied from 0.152–0.891 and values for the ratio T_b/T_p varied from 2.55–4.88.

TR-55 Method.—Chapter 7 of Technical Release No. 55, or TR-55 (SCS, 9), provides a method "for quickly analyzing effects of storage reservoir on peak discharges." It is based on average storage and routing effects for many structures that were evaluated using the more accurate TR-20 method (SCS, 10). Two relationships are provided. For peak flows through pipe spillways that are less than 300 cfs/square mile of drainage area (CSM) the detention storage in watershed inches is estimated using the volume of runoff in watershed inches and the release rate. For peak flows through pipe spillways that are greater than 300 CSM, the ratio of the volume of storage to the volume of runoff (V_s/V_r) is given as a function of the ratio of the peak rate of outflow to the peak rate of inflow, Q_0/Q_i.

Loss of Natural Storage Method.—The physical changes that accompany urban

development decreases the natural storage within the watershed. Specifically, interception and depression storage are decreased, and because the spatial potential for infiltration is decreased, subsurface storage is also decreased. Stormwater management basins are an attempt to replace the natural storage that is lost because of development with man-made storage. Thus, the difference between the natural storage before and after development can be used as an estimate of the man-made storage that is required to meet the intent of stormwater management. A hydrologic method that determines the volume of runoff before and after development is all that is required to evaluate the required volume of storage; for a given event many such methods are available as it is not necessary to compute hydrographs.

Generalized Model for Estimating Required Storage

An estimate of the required detention storage is necessary in both planning and design. Planning methods are usually based on one or more of the following

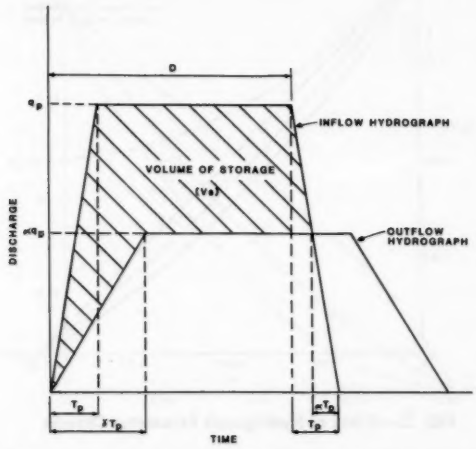

FIG. 1.—Conceptual Representation of Inflow and Outflow Hydrographs

parameters: (1) α: the ratio of the peak discharges before development (i.e., the peak discharge from the detention facility) and after development; (2) γ: the ratio of the times-to-peak for the flow out of and into the detention facility; (3) β: the ratio of the time base of the hydrograph, Tb, to the time-to-peak (T_p); and (4) V_r: the volume of runoff after development. Before considering the assumptions that are made for each method about these parameters, it is instructive to present a generalized model for relating these parameters.

Generalized Planning Model.—Fig. 1 shows estimated hydrographs of inflow to and outflow from a SWM basin. Using the geometric relationships between the hydrograph characteristics, the following generalized planning model was derived:

$$\frac{V_s}{V_r} = 1 - \alpha\left[1 + \frac{T_p}{D}\left(1 - \frac{1}{2}(\gamma + \alpha)\right)\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which V_s = the volume of storage; V_r = the volume of runoff; and D = the duration of precipitation. Fig. 2 shows the effect of varying the input parameters α, γ, and T_p/D, on the ratio of the volume of storage and the volume of runoff after development.

Comparison with Other Planning Models.—It is of interest to compare the generalized equation with the other planning methods. The relationship between V_s/V_r and α provides one means of comparison. To facilitate comparison it is necessary to make several simplifying assumptions. A triangular hydrograph with a time base, T_b, equal to twice the time-to-peak, T_p, is not an unreasonable assumption given the frequency with which the rational method is used in

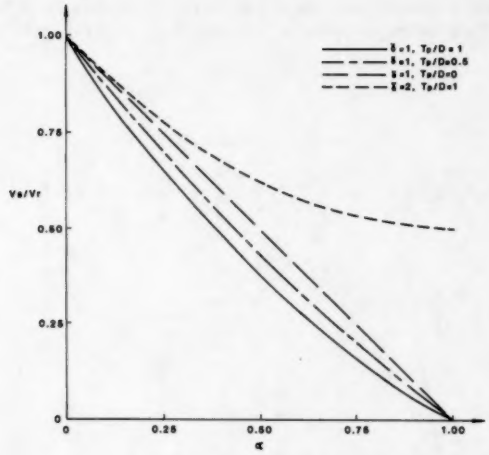

FIG. 2.—Effect of Hydrograph Parameter Change

hydrologic analyses; it will also be assumed that the time-to-peak equals the time of concentration, t_c. Based on these assumptions and the assumption that the volume of runoff equals the product of the peak discharge and the time of concentration, i.e., $V_r = T_p Q_p$, the Abt and Grigg model, Eq. 1, can be rearranged to give:

$$\frac{V_s}{V_r} = (1 - \alpha)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

For the triangular hydrograph $T_b/T_p = 2$; therefore, the Wycoff and Singh model (Eq. 2) reduces to:

$$\frac{V_s}{V_r} = 0.97(1 - \alpha)^{0.753} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

The generalized planning model (Eq. 3) reduces to:

$$\frac{V_s}{V_r} = 0.5(1-\alpha)(2-\alpha) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

For pipe flow structures and discharges greater than 300 CSM, the TR-55 relationship is given by:

$$\frac{V_s}{V_r} = 0.28477\alpha^{-0.32366} \qquad 0.1 < \alpha < 0.35 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7a)$$

$$\frac{V_s}{V_r} = 0.1420\alpha^{-0.98625} \qquad 0.35 \leq \alpha \leq 0.8 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7b)$$

A comparison of Eqs. 4, 5, 6, and 7 are shown in Fig. 3. The maximum differences between the four methods occur for values of α between 0.5 and 0.75, although

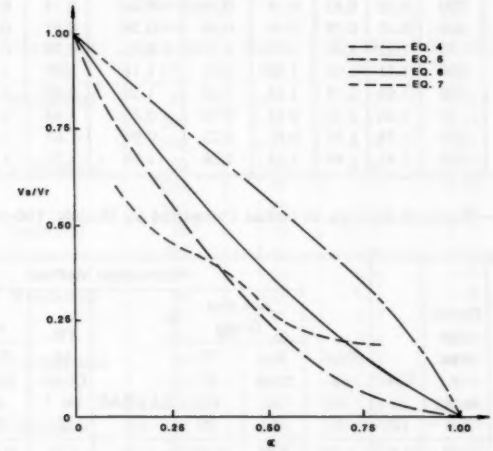

FIG. 3.—Comparison of Methods for Triangular Inflow Hydrograph

the TR-55 method shows significant deviation from the other three methods for values of α below 0.25. A description of the significance and reasons for these differences will be given in the following.

Case Study Comparison of Methods

Recognizing that the methods are based on different assumptions, it is not unreasonable to assume that the methods will give different answers and will differ in accuracy. Accuracy, which is a measure of the difference between the estimated value and the true value (McCuen, 8), cannot be assessed for analyses involving stormwater management because the data required to determine

the true value is not available. In fact, the lack of SWM data collection programs precludes even an empirical assessment of accuracy. Because of this unavailability of SWM data it is necessary to use hypothetical case studies to compare the methods. Actually, this requirement may provide a better comparison, even if accuracy cannot be assessed, because the watershed and precipitation charac-

TABLE 1.—Required Storage, in inches Computed by Models: 10-yr Design

Land use (1)	Drainage area, in acres (2)	Baker (3)	Bondelid (4)	Abt and Grigg		ILLUDAS (7)	TR-55 Chapter 7 (8)	Wycoff and Singh	
				Rational (5)	TR-55 (D) (6)			Rational (9)	TR-55 (D) (10)
Residential	50	0.47	0.51	0.29	0.30	0.34	0.72	0.46	1.10
	200	0.58	0.55	0.38	0.34	0.46	0.74	0.56	1.10
	500	0.63	0.58	0.46	0.38	0.58	0.91	0.64	1.10
Commercial	50	1.22	1.44	0.79	1.15	0.96	1.93	0.93	2.78
	200	1.51	1.63	1.08	1.53	1.11	1.97	1.27	2.83
	500	1.68	1.78	1.23	1.69	1.26	2.02	1.42	2.83
Industrial	50	1.02	1.23	0.68	0.96	0.83	1.64	0.82	2.36
	200	1.24	1.38	0.89	1.23	0.94	1.67	1.06	2.39
	500	1.43	1.49	1.04	1.36	1.08	1.71	1.23	2.39

TABLE 2.—Required Storage, in inches Computed by Models: 100-yr Design

Land use (1)	Drainage area, in acres (2)	Baker (3)	Bondelid (4)	Hydrologic Method					
				Abt and Grigg		ILLUDAS (7)	TR-55 Chapter 7 (8)	Wycoff and Singh	
				Rational (5)	TR-55 (D) (6)			Rational (9)	TR-55 (D) (10)
Residential	50	0.71	1.00	0.42	0.40	0.25	1.22	0.67	1.82
	200	0.83	1.14	0.49	0.46	0.58	1.24	0.73	1.82
	500	0.81	1.25	0.68	0.51	0.86	1.24	0.96	1.82
Commercial	50	1.84	2.25	1.13	1.51	1.10	2.83	1.34	4.13
	200	2.08	2.59	1.60	2.34	1.27	2.88	1.89	5.10
	500	2.09	3.10	1.86	2.26	1.62	2.90	2.15	4.24
Industrial	50	1.60	2.00	0.97	1.27	0.94	2.52	1.19	3.59
	200	1.76	2.29	1.31	1.66	1.16	2.56	1.59	3.68
	500	1.75	2.60	1.57	1.83	1.43	2.57	1.88	3.68

teristics can be controlled. Although the accuracy cannot be determined, the objective of the comparison is to identify the relative differences between methods, so regulatory agencies can decide whether or not they should place restrictions on the methods that can be used in the planning and design of SWM basins.

Framework for Comparison.—To make comparisons of the methods, watershed and precipitation characteristics that are typical of the Piedmont Plateau in Maryland were used. Hypothetical watersheds, with drainage areas of 50 acres, 200 acres, and 500 acres, were developed. A grass covered land use was assumed to be the predevelopment conditions; this was represented by an SCS curve number of 65 and a rational method coefficient of 0.2. Slopes of 2.5% were assessed. The SCS Soil Group B was used. For the after-development conditions three land uses were included: (1) Residential; (2) commercial; and (3) industrial (homogeneous patterns of development were assumed). The corresponding rational method coefficients are 0.4, 0.8, and 0.7; the SCS runoff curve numbers were 75, 92, and 88, respectively, for residential, commercial, and industrial

TABLE 3.—Computed Volumes of Effective Natural Storage, in inches Lost by Development

Return period (1)	Land use (2)	Drainage area, in acres (3)	Baker (4)	ILLUDAS (5)	Rational (6)	TR-55 (D) (7)
10 yr	Residential	50	0.39	0.37	0.22	0.77
		200	0.37	0.37	0.27	0.77
		500	0.57	0.37	0.32	0.77
	Commercial	50	1.36	0.84	0.65	2.37
		200	1.70	1.15	0.98	2.37
		500	1.75	1.15	1.11	2.37
	Industrial	50	1.09	0.71	0.55	1.96
		200	1.42	0.96	0.77	1.96
		500	1.54	0.96	0.93	1.96
100 yr	Residential	50	0.56	0.25	0.30	1.07
		200	0.56	0.53	0.34	1.07
		500	0.60	0.54	0.46	1.07
	Commercial	50	2.05	0.72	0.90	3.00
		200	2.09	1.25	1.45	3.00
		500	2.27	1.26	1.66	3.00
	Industrial	50	1.74	0.59	0.77	2.53
		200	1.83	1.06	1.12	2.53
		500	1.85	1.06	1.39	2.53

land uses. The percentages of paved area were 34%, 83%, and 70%, respectively. Stormwater runoff was assumed to flow across developed areas as overland flow to roadway gutters, which are discharged through a single outlet at the end of the basin. Overland flow conditions exist for the predeveloped drainage area for the first 400 ft of flow path where it then flows through a grassed ditch to the basin outflow. The travel times for the overland flow portion are computed using Izzard's method in the IDDUDAS program and the Maryland State Highway nomograph for the remaining methods, while the channelized flow travel times both for grassed gutters and roadway gutters are determined using Manning's equation. Analyses were made for precipitation volumes having return periods of 10 yr and 100 yr; all methods assume that the return period of the runoff is the same as that for the precipitation. For the TR-55 and

TR-20 based methods, a 24-h precipitation is assumed. For the ILLUDAS method, rainfall durations of 0.5 h, 1.0 h, and t_c were tested. For the methods based on the rational method, the critical storm duration (i.e., the storm duration resulting in the maximum detention volume) was determined and the volumes were computed with this storm duration.

Comparative Results.—Precipitation for exceedence probabilities of 0.1 and 0.01 (i.e., 10 yr and 100 yr) were used as input with the 9 land use/drainage area combinations. For the Abt and Grigg (1) and Wycoff and Singh (12) models runoff computations were made with both the rational formula and the TR-55 chart method. Tables 1 and 2 show the estimated storage required for each method. Table 3 shows the computed volumes of effective natural storage that was lost by development; the losses of natural storage are shown for four methods: (1) The Baker method; (2) the Rational formula; (3) The ILLUDAS; and (4) the TR-55 Chart method. The resulting volumes of required storage show considerable variation. For example, the values ranged from 0.22 in.–1.10 in. for the 50 acre residential development case study with a 10-yr precipitation.

TABLE 4.—Ranges of Computed Storages for Planning and Design Methods: 10-Yr Event

Land use (1)	Drainage area, in acres (2)	Design methods, in inches (3)	Planning methods, in inches (4)
Residential	50	0.47–0.51	0.22–1.10
	200	0.55–0.58	0.27–1.10
	500	0.58–0.63	0.32–1.10
Commercial	50	1.22–1.44	0.65–2.78
	200	1.51–1.63	0.98–2.83
	500	1.68–1.78	1.11–2.83
Industrial	50	1.02–1.23	0.55–2.36
	200	1.24–1.38	0.77–2.39
	500	1.43–1.49	0.93–2.39

For the 500-acre commercial case, the storage required to control the 100-yr precipitation ranged from 1.11 in.–4.24 in., or 46.3 acre-ft–176.7 acre-ft. These represent very significant differences with respect to both planning and design and indicate that careful consideration should be given to selection of a method for estimating the required volume of storage.

It is of interest to compare the range of the values of required storage for both planning and design methods. The Baker method and Bondelid method are used to represent the design methods while the remaining methods represent planning methods. The design methods show surprising agreement in spite of their quite different conceptualization. The planning methods show considerable variation. It is also important to note that the values for the design methods fall near the median of the planning methods (Table 4).

Comparison of Detention Storage Designs

Effect of Method Assumptions on Storage Volume.—The wide range in required storage volumes results from the effect of both hydrologic and hydraulic

considerations. It is important to separate the affect of these two considerations in order to evaluate their relative importance when comparing stormwater management methods. The general equation, Eq. 3, can be solved to find the rainfall duration that gives the maximum volume of storage for any given return period. This critical storm duration is a function of both the hydrologic parameters and precipitation characteristics.

The rainfall intensity, volume of runoff, and peak runoff can be expressed in terms of rainfall duration and some constants. From rainfall-intensity-duration curves it can be shown for storms of duration less than 2 hr:

$$I = \frac{a}{b + D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

in which I = rainfall intensity; D = rainfall duration; and a and b are curve coefficients. From Fig. 1 the volume of runoff after development is:

$$V_r = q_p D \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Using the Rational method for finding peak runoff and by substituting Eq. 8 into Eq. 9, the volume of runoff becomes:

$$V_r = C\left(\frac{a}{b + D}\right) AD \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

in which C = runoff coefficient of the method; and A = drainage area. Using the same reasoning the ratio of peak runoff before development to peak runoff after development becomes:

$$\alpha = \frac{q_p(b + D)}{CaA} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Substituting Eqs. 10 and 11 into Eq. 3 gives the volume of storage in terms of rainfall duration and watershed constants:

$$V_s = \frac{0.5 t_c q_p^2 (b + D)}{CaA} - q_p D - t_c q_p + 0.5\gamma t_c q_p + \frac{CaAD}{b + D} \quad \ldots\ldots\ldots\ldots (12)$$

By using the Rational Method, the maximum inflow peak occurs either at the time of concentration or less. At a time longer than the time of concentration, the inflow peak decreases but the volume of runoff increases. Depending upon the hydrologic parameters of a given case, the maximum volume of detention storage will occur at some time equal to or exceeding the time of concentration. A plot of V_s from Eq. 12 versus the duration (D) would give a curve with a maximum duration, D_{max}. By taking the first derivative of Eq. 12, setting it equal to zero and solving for D, the critical duration equal to maximum detention is:

$$D_{max} = \left[\frac{CaAb}{q_p - \dfrac{0.5 t_c q_p^2}{CaA}}\right]^{1/2} - b \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

Having determined V_s without considering the hydraulic parameters of the

detention basin, the general equation becomes a planning equation for the Baker method. The planning equation is solved by finding the critical duration from Eq. 13 and then using Eq. 12 for determining V_s.

The variation in volume of storage for four cases using the Baker design model and planning equation is illustrated in Table 5. The general equation

TABLE 5.—Comparison of Results from Design Model and Planning Equations using Baker Method

Case (1)	Mode (2)	Critical storm duration, in hours (3)	After development peak flow, in cubic feet per second (4)	Volume of runoff, in inches (5)	Volume of detention storage, in inches (6)
10 yr-50 acre	Design	0.68	60	0.80	0.47
Residential	Planning	0.48	76	0.73	0.49
10 yr-50 acre	Design	1.05	87	1.77	1.22
Commercial	Planning	0.76	111	1.69	1.31
100 yr-500 acre	Design	1.83	861	3.03	2.09
Commercial	Planning	1.31	1,137	2.98	2.46
100 yr-500 acre	Design	1.08	660	1.36	0.81
Residential	Planning	0.85	790	1.34	1.05

TABLE 6.—Comparison of Pipe Spillway Diameters from Baker and Bondelid Models

	BAKER MODEL				BONDELID MODEL		
	10-yr		100-yr		Control for both 10-yr and 100-yr		
Watershed (1)	Riser, in feet (2)	Conduit, in feet (3)	Riser, in feet (4)	Conduit, in feet (5)	Riser, in feet (6)	Conduit, in feet (67)	Orifice, in feet (8)
Residential							
50 acres	2.00	1.50	2.50	1.75	3.75	2.50	1.75
200 acres	2.50	2.00	3.00	2.50	5.50	3.50	2.75
500 acres	3.00	2.50	4.50	3.50	8.00	4.50	3.75
Commercial							
50 acres	2.00	1.50	2.50	1.75	3.75	2.50	1.75
200 acres	2.50	2.00	3.00	2.50	8.00	3.50	2.75
500 acres	3.00	2.50	4.50	3.50	8.00	4.50	3.75
Industrial							
50 acres	2.00	1.50	2.50	1.75	3.75	2.50	1.75
200 acres	2.50	2.00	3.00	2.50	8.00	3.50	2.75
500 acres	3.00	2.50	4.50	3.50	8.00	4.25	3.75

gives consistently larger volumes of storage for each case. It is important to note that the variation between planning estimates made with the generalized equation and estimates made with the Baker method is small relative to the variation between planning methods. These results suggest that the wide range

in volume of storage is largely caused by the hydrologic assumptions made in each method rather than by the effect of detention basin hydraulics.

Comparison of Pipe Spillway Designs.—The two design models, Baker and Bondelid, use pipe spillways as the hydraulic outlet for the stormwater detention basin. The Bondelid method designs a single structure to control runoff from two design storms. The discharge from the more frequent storm, in this case the 10-yr storm, is limited by the orifice diameter nearer the base of the structure. The riser height and diameter is the controlling factor for the upper limit design storm. The pipe drop inlet spillway used for the Baker method is designed to control the discharge from a design storm of a single return period by limiting the basin discharge through the conduit. Comparison of the selected spillway sizes for the two methods is shown in Table 6. It should be noted that the pipe sizes shown in Table 6, while representative of those required for a particular storm event, are not unique. A range of pipe sizes can be used depending on the geometry of a particular spillway and detention basin.

Review

Terminology.—The term "stormwater management" is a term that has evolved because it is recognized that development most often increases the potential for damage from flood runoff. A number of management techniques have been used to mitigate the hydrologic effects of increased development, with both structural and nonstructural methods being used. Structural techniques are construction related while nonstructural include legal and administrative procedures. A structure that creates a reservoir is the most popular method of controlling runoff. A number of terms have been applied to these structures, including onsite ponds, detention basins, retention storage, and stormwater management basins, with the latter term being used herein. The term "detention" implies a relatively short time period for detaining stormwater runoff while the term "retention" implies a relatively longer time period; however, it is not always clear how these two differ. In some cases, a "wet" pond is referred to as a retention structure while a "dry" pond refers to a detention pond; a wet pond is one in which some storage is provided in which losses occur only by evaporation and infiltration. "On-site ponding" is a general term that can include both detention and retention ponding. The term "stormwater management basin," which is the most general, can refer to both surface or subsurface storage. It is frequently used, as it was herein, because it avoids the ambiguities of the other terms; also, the methods described herein could be applied in determining the required storage of both detention and retention structures.

Effect of Variation in Conceptual Development.—The methods give significantly different estimates for a variety of reasons. First, several of the methods were based on storm durations of 24 h while others used durations of 1 h and less. Storm duration can have a significant effect on the required storage and the peak reduction factor (McCuen, 7). The effect of duration on the required volume is interdependent with the change in land use and the hydrologic method. Second, the methods varied with respect to their conceptualization of the runoff process. For example, the Abt and Grigg method assumed that the rising sides of the inflow and outflow hydrographs coincided up to the time when the outflow rate was limited; other methods did not make this assumption. Those methods

that assume the rising limbs coincide will have smaller volumes of storage. The differences in conceptualization between the rational method and the SCS runoff curve number approach also lead to differences in estimated storages. Third, some of the methods assumed triangular or trapezoidal hydrographs while others were based on curvilinear hydrographs. Fourth, two methods included hydraulic routing of flow through the basin outlet while the other methods did not provide for routing. In summary, the methods give different estimates because of differences in the underlying solution methodology.

Accuracy.—While the large differences in required storage may be illuminating to those involved in approving designs, it is very unfortunate that the accuracy of each method cannot be assessed. In order to quantify the accuracy of the methods, it would be necessary to know the true required volume of storage that would meet the intent of stormwater management. Accuracy equals the expected difference between the estimated and true values (McCuen, 8). Since the true required storage is indeterminant, the accuracy of each method cannot be assessed. The implication of the inability to estimate accuracy is that a method must be selected when, in fact, there is no way of knowing whether or not it provides the best estimates.

Considerations in Selecting Method.—Recognizing that different methods give significantly different estimates and that it is not possible to determine which method is most accurate, it is still necessary to select a method. In order to select a method, the criteria for selection should be identified by the regulatory agency. For example, the degree of hazard downstream from the site of development must be assessed. In high hazard areas, it may be preferable to use a method that gives high estimates. Another consideration is the computational requirements, which includes both the data input requirements and the requirement for a computer. A method that does not require a digital computer may be preferable because such facilities may not be readily available to all those involved in design of stormwater management basins. If there exists a policy that specifies a certain method for computing runoff (e.g., the rational method, the SCS TR-55 method) must be used, then it may be preferable to use a method for estimating SWM basin volumes that is based on similar hydrologic concepts; this will eliminate the need to collect excessive input data. Furthermore, if different methods are to be used for planning and design, the two methods should be based on a similar conceptualization of the runoff process. In summary, stormwater management policies should be properly integrated with any existing policy on estimating runoff volumes and rates.

In selecting a method, one must consider the impacts of both over-design and under-design. If a method consistently provides less storage volume than is really needed, then the intent of stormwater management will not be met; this will lead to excessive downstream flooding and inadequate control of water quality pollutants, including sediment. Inadequate storage may also lead to degradation of the channel downstream because of the increase in both discharge rates and the duration of high flows. Over-design requires an excessive allocation of land for the SWM basins. This represents a direct economic loss to the developer. Furthermore, the lower level of development may have an adverse effect on the potential tax base. In summary, there are important implications in the selection of a planning/design method, and it has been shown that the available methods provide significantly different estimates.

Recommendation.—Because it was not possible to determine the accuracy of the methods, the obvious criterion cannot be used to recommend one method. Thus, a method can be selected only on the basis of the underlying model characteristics. The authors recommend the use of the Bondelid method in design situations for numerous reasons. First, it uses a 24-h storm duration, and the SCS storm distributions were designed to include peak intensities for durations as small as 1 hr. Second, the method provides for designs based on two exceedance probabilities (i.e., return periods). Third, the method determines several feasible designs which allows the design engineer to select the most practical design. Fourth, the method determines the volume of storage and the pipe spillway characteristics simultaneously. Fifth, the method provides for hydrograph routing through the spillway. Sixth, the method is based on a hydrologic procedure (i.e., the Soil Conservation Service's TR-20) that has been used extensively for hydrologic computations.

In most cases the planning models gave significantly different estimates of the computed storage than did the Bondelid method. However, the TR-55 method, which is a planning model, gave estimates within reasonable accuracy as compared with the Bondelid method. Furthermore, it requires input similar to that of the Bondelid method, and thus, use of the TR-55 method for planning would not require a totally new set of input data to be collected for the design phase. Therefore, the writers recommend the use of the TR-55 method for planning purposes.

Conclusions

A number of methods that are used for either planning or design (or both) of storm-water management basins were evaluated and compared. Significant differences in the conceptual development of the methods were identified. Comparisons of the methods for different land uses (i.e., residential, commercial, and industrial) and various drainage areas (50 acres, 200 acres, and 500 acres) showed that the methods give significantly different estimates of the required volume of storage.

The significance of the differences in storage computed by the various methods is most evident from a consideration of the volumes of precipitation and runoff. In the Washington, D.C. area, the 1-h and 24-h precipitation volumes are 2.25 in. and 4.3 in., respectively, for a 10-yr frequency. While the volume of runoff will depend on various factors including soils, topography, land use and antecedent moisture, it is not unreasonable to expect that the runoff volumes could be on the order of 1 in. and 2 in. for the 1-h and 24-h storms, respectively. The values in Table 1 indicate that the required volumes of storage are a very significant percentage of the total runoff; furthermore, the values indicate that the percentage varies considerably from one method to the next. Therefore, it is reasonable to assume that the methods give significantly different estimates and that it may be prudent for regulatory agencies to select one method that best represents the hydrology of the locality.

Appendix.—References

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In-Home Conservation and Wastewater Management

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Introduction

The Federal Water Pollution Control Act Amendments of 1972 (PL 92-500) as amended by the Clean Water Act of 1977 (PL 95-217) require that all publicy-owned municipal wastewater treatment plants provide best practicable wastewater treatment technology (BPWTT) over the life of the facility. This has been defined to include a requirement for a minimum level of secondary treatment by July 1, 1977, except where receiving-water quality considerations dictate more stringent levels of treatment.

To achieve this end, the two acts authorize approx $45,000,000,000 in federal grants to states and municipalities to pay 75% of the cost of constructing collector and interceptor sewers and wastewater treatment plants (5). Under the construction grants program, the Environmental Protection Agency (EPA) has already distributed approx $19,500,000,000 to finance construction of $26,000,000,000 worth of municipal wastewater treatment facilities (18). Current estimates indicate that an additional $95,900,000,000 worth of new construction will be required by 1990 to meet BPWTT goals. Approx $34,200,000,000 (35.7% of the total) will finance constructing new treatment plants as well as upgrading or enlarging, or both, existing treatment plants (7). By 1990, approx 1.578×10^8 m^3/day (41.7 BGD) will be discharged from municipal wastewater treatment plants providing at least secondary treatment. Most of this treatment capacity will be provided by newly-built, existing or enlarged or upgraded, or both, activated-sludge plants.

In the face of the large expenditures associated with constructing and upgrading wastewater treatment plants (8,9,10), EPA has issued regulations requiring a cost-effectiveness analysis of alternative municipal wastewater treatment processes for every project to be funded under the municipal construction grants program (12). Guidelines for the cost-effectiveness analysis generally require consideration of the costs, cost-savings, and effects of flow reduction measures,

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as they apply to both industrial sources and households. Cost-effectiveness analysis of household flow reduction is the focus of this study.

Household Water Conservation

In many cities, nearly all of the water supplied to private households finds its way into the municipal wastewater treatment system. Household water conservation is, therefore, often an important component of municipal wastewater flow reduction. Table 1 provides estimates of household water use, and Table 2 lists the costs and water savings associated with installing various household water conservation devices.

TABLE 1.—Typical Daily Use of Household Water in Family of Four

Usages (1)	Gallons per family per day (2)	Gallons per capita per day (3)	Liters per family per day (4)	Liters per capita per day (5)	Percent of total (6)
Toilet (5 flushes/person × 5 gal/flush × 4 persons)	100	25	378.5	94.6	39
Bathing (5 gal/min × 4 min/shower × 1 shower/person day × 4 persons)	80	20	302.8	75.7	31
Laundry (50 gal/load × 0.7 load/day)	35	8.75	132.5	33.1	14
Dishwasher (15 gal/load × 1 load/day)	15	3.75	56.8	14.2	6
Kitchen sink	12	3	45.4	11.4	5
Lavatory	8	2	30.3	7.6	3
Utility sink	5	1.25	18.9	4.7	2
Totals	255	63.75	965.2	241.3	100

Note: Source References (1,2,15).

TABLE 2.—Potential Household Water Conservation Devices for Use in Existing Homes

Fixture (1)	Device (2)	Cost/device (3)	Percent reduction in total household water use/ wastewater flow (4)
Toilet	Plastic bottles, dams	$0–$6	5–20
Shower	Low flow shower heads, flow restrictors	$1–$25	5–15
Sinks	Low flow aerators	$1–$5	2
—	Pressure reducing valves	$25	5
—	Insulated hot water pipes	$0.50/ft of line	1

Although the study focuses on the cost-effectiveness of reducing the flow of municipal wastewater from households by installing water conservation devices, two other considerations need to be mentioned, even though they were not specifically included: Public education programs are an important household water conservation measure. By modifying ordinary behavior, large volumes of water can be conserved and thus wastewater reduced. Examples include not letting the water run while brushing one's teeth or while shaving; turning off the shower while lathering up.

This study focuses on water conservation by installing specific devices which become transparent to the user, and which are provided to the user without charge. While behavior changes are important and have been effective, the likelihood for implementation and continued use of a device is probably greater than that of a measure which requires behavior changes. The second consideration is that household water conservation measures save more than the cost of providing treatment for the resulting municipal wastewater. In most cases, household water conservation saves the marginal cost of treating the water supply to potable quality in the first place. If the water savings includes hot water, then a substantial energy savings occurs. Neither of these savings is included in this analysis.

If the savings in wastewater treatment costs exceeds the costs of achieving the reduction, then household water conservation is a cost-effective component in a wastewater treatment strategy. This is a very conservative estimate of the overall cost-effectiveness of water conservation because it includes all the costs of conservation but neglects the savings in the costs of water supply and water heating.

Effects of Household Water Conservation

Fig. 1 is helpful in showing the manner in which household water conservation results in reducing wastewater treatment costs. The solid line represents the flow of wastewater from a community of 600,000, growing at 2%/yr, assuming that total wastewater flow is an average of 0.549 m^3/cap·day (145 gal/cap·day). The dashed line represents this same flow reduced by 10%. The horizontal line, Q_{max}, is the existing treatment capacity in year zero. The total volume of wastewater treated over any interval of time can be calculated as the area under the flow curve.

If a conservation program is implemented to reduce the total wastewater flow by 10%, the flow changes from the solid to the dashed flow curve. The area under the curve (the volume of wastewater) is reduced. This would result in a corresponding reduction in those operations and maintenance (O&M) costs that are flow-dependent. The total reduction in volume, and thus in O&M costs, over the entire 30-yr period depends on the actual percent reduction in flow and the year in which the reduction in flow (i.e., the change from the solid to the dashed line in Fig. 1) occurs. The more and the earlier the wastewater flow is reduced, the greater the total O&M cost savings to the wastewater treatment utility.

If construction of additional treatment facilities is required when wastewater flow reaches the existing capacity, then flow reduction will affect the construction cost in two ways. First, since capacity will be reached later (in year 17, as

opposed to year 12, in Fig. 1) because of the smaller flow, construction of new facilities can be postponed. The present value of this capital outlay decreases as it is moved into the future, as determined by the discount rate of the wastewater treatment utility. Thus, delaying the cost of construction of a new plant will result in a present value savings, provided that inflation does not increase the construction cost at a rate rapid enough to offset the savings due to discounting.

Second, if the utility needs to provide adequate treatment capacity for a specific time horizon (e.g., to year 30 in Fig. 1) then the expanded treatment facilities can be designed to handle a smaller flow if household water conservation measures are implemented. However, the wastewater from a community that has implemented water conservation measures will be of a higher strength than the wastewater from an identical community that has not reduced water use. Specifically, concentrations of five-day biochemical oxygen demand (BOD_5) and total suspended solids (TSS) in the treatment plant influent will be increased

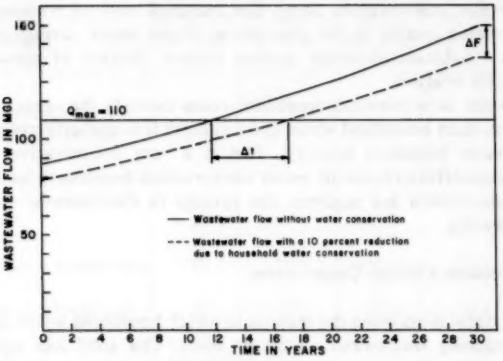
160
100
50
ΔF
Q_{max} = 110
Δt
WASTEWATER FLOW IN MGD
Wastewater flow without water conservation
Wastewater flow with a 10 percent reduction due to household water conservation
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30
TIME IN YEARS

FIG. 1.—Effect of Conservation on Time-Dependence of Wastewater Flow

because water conservation reduces the volume of water, but does not reduce the mass of organic matter present.

The current definition of secondary treatment, for purposes of meeting federal requirements, is effluent concentrations not to exceed 30 mg/L either for BOD_5 or for TSS (when influent concentrations exceed 200 mg/L) (6). By defining secondary treatment in terms of effluent concentrations, EPA regulations require a higher removal efficiency at the treatment plant for a wastewater treatment strategy that incorporates water conservation measures than for one that does not. The removal efficiency for activated sludge plants has been shown virtually constant over a reasonable range of influent flow rates and concentrations (3). Therefore, applying water conservation measures will require design or operational (or both) changes in the treatment plant to achieve the higher removal rate needed to meet federal effluent concentration standards for secondary treatment. This will result in a reduction in the mass loading entering the receiving stream.

Incorporating water conservation measures in a wastewater treatment strategy

results in an additional environmental benefit beyond conserving resources. This is illustrated in Table 3 for the same community used in Fig. 1. However, this means that the net construction cost savings due to water conservation will be the difference between the cost savings from reducing the design flow of the plant and the cost increase from necessarily increasing the removal efficiency.

The decrease in design flow for the new plant, as well as the delay in construction, depend only on the amount by which the flow is reduced and not on the year in which the reduction takes place, provided the flow is reduced before present capacity is reached. This can be seen in Fig. 1, where a 10% flow reduction is essentially a discontinuous jump from the solid to the dashed flow curve. Once this jump occurs, the per capita rate of wastewater generation is reduced to 0.494 m^3/cap·day (130.5 gal/cap·day). Thereafter, the reduced wastewater flow is 90% of the original flow. As long as the jump takes place before Q_{max} is reached, Δt, and ΔF in year 30 will be the same, regardless of the year in which the jump takes place.

TABLE 3.—Impact of Water Conservation on Wastewater Treatment Plant Efficiency

Item (1)	Without water conservation (2)	With 10% water conservation (3)
Volumetric flow rate, 10^4 m^3/day	32.93	29.64
Influent BOD_5 and TSS concentrations, in milligrams per liter	200.	222.
Effluent BOD_5 and TSS concentrations required to meet secondary treatment standards, in milligrams per liter	30.	30.
Required treatment plant removal efficiency, percent	85.	86.5
Daily BOD_5 and TSS loads entering receiving stream, in kilograms per day	9,874.	8,886.

Assuming a constant per capita flow reduction in a growing community implies reduction of the flow generated by persons who move into the community after the initial household water conservation measures are implemented. This means that any conservation devices which are used in households to achieve flow reduction must be provided for the entire population that will be in the community in the final year of the designated period, rather than just the population at the time the conservation program is initiated. The cost of the conservation program, then, must be based on this higher number of devices.

Development of Model

A mathematical model was developed to quantify the above considerations concerning the cost-effectiveness of household water conservation as a component of municipal wastewater treatment. The model estimates the savings in O&M and construction costs which could result from a household water conservation

program and compares the savings to the costs involved in implementing the program. Description of the model is presented in three sections: (1) The first considers the O&M cost savings; (2) the second considers the construction cost savings; and (3) the third considers the cost of the household water conservation program.

O&M Cost Savings

Because implementing household water conservation measures both decreases the volume of wastewater to be treated and increases the required treatment efficiency, a literature search was undertaken to obtain a method for estimating O&M costs as a function of these two parameters. A recent EPA study (10) considered that relationship, but the results did not appear statistically significant, based upon the sample analyzed. In 1970, a study was published to relate O&M costs of wastewater treatment separately to the population equivalent treated and to the average daily flow treated (16). Combining these relationships for conventional activated-sludge treatment plants and updating to reflect second quarter 1977 costs produces extremely close agreement with estimates from the recent EPA study (10). The following equation was estimated:

$$OM = 872\,(P_e Q)^{0.488} \quad \ldots\ldots (1)$$

in which OM = annual O&M cost, in dollars; P_e = population equivalent treated by the plant; and Q = average daily flow, in MGD (1 MGD = 3,785 m^3/day).

If population grows exponentially at $100k\%$/yr, and wastewater is generated at a constant rate of C gal/cap·day (1 gal/cap·day = 3.785×10^{-3} m^3/cap·day) then population and average daily flow of wastewater can be written as:

$$P = P_0 e^{kt} \quad \ldots\ldots (2)$$

$$Q = \left(\frac{CP_0}{10^6}\right) e^{kt} \quad \ldots\ldots (3)$$

in which P_0 = initial population, in persons, and t = time, in years.

If residential flow accounts for a constant $100b\%$ of the total influent to the municipal wastewater treatment plant over time, with the remaining flow accounted for by industrial/commercial sources, then the total population equivalent (P_e) treated by the plant can be estimated as:

$$P_e = \left(\frac{P_0}{b}\right) e^{kt} \quad \ldots\ldots (4)$$

Substituting Eqs. 3 and 4 into Eq. 1 gives:

$$OM = Be^{0.976kt} \quad \ldots\ldots (5)$$

$$\text{in which} \quad B = 1.029\,P_0^{0.976}\left(\frac{C}{b}\right)^{0.488} \quad \ldots\ldots (6)$$

If inflation increases annual O&M costs at $100i_1\%$/yr and the municipal wastewater treatment utility discounts future costs at $100r\%$/yr, then the effective interest rate for O&M costs, including both inflation and discounting is (14):

$$I_r = r - i_1 - ri_1 \quad \ldots \quad (7)$$

in which i_r = effective interest rate for O&M costs. Assuming continuous compounding, O&M costs from Eq. 5 can be discounted at the equivalent interest rate to obtain the present equivalent, POM, as follows:

$$\text{POM} = Be^{Et} \quad \ldots \quad (8)$$

in which $E = 0.976k - I_r$.

Integrating Eq. 8 to sum the annual costs over the time period from year x to year T and to obtain the net present value of the series of O&M costs, TPOM, gives Eq. 9:

$$\text{TPOM} = \left(\frac{B}{E}\right)(e^{Et} - e^{Ex}) \quad \ldots \quad (9)$$

If a conservation measure is implemented that will reduce the wastewater flow from any household by an average of $100s\%$, $100a\%$ of the households actually adopt and use the water conservation measure, and $100b\%$ of the total wastewater flow is from households, then the total wastewater flow is reduced by $100abs\%$. Under these conditions, the average daily flow is reduced to $Q(1 - abs)$. Eqs. 5 and 9 can be modified to account for the reduced average daily flow, keeping the population equivalent constant to approximate treatment to meet the same effluent concentration standards. This procedure results in the following:

$$\text{TPOM}' = \text{TPOM}(1 - abs)^{0.488} \quad \ldots \quad (10)$$

in which TPOM = total net present value of all the O&M costs from year x to year T, in dollars, given application of the specified household water conservation measure. The net present value of the O&M costs savings due to household water conservation measures, OSAV, is therefore:

$$\text{OSAV} = \text{TPOM}\,[1 - (1 - abs)^{0.488}] \quad \ldots \quad (11)$$

Construction Cost Savings

A literature search revealed a 1970 study which related the construction cost of municipal wastewater treatment plants to the design population equivalent and the design flow to be handled by the plant (17). The relationship for conventional activated-sludge treatment plants was modified and updated to reflect second quarter 1977 costs, and resulted in very close agreement with a recent EPA study (9). The resulting equation is:

$$CC = 52{,}500\, P_D^{0.461}\, Q_D^{0.429} \quad \ldots \quad (12)$$

in which CC = construction cost, in dollars; Q_D = design flow, in MGD (1 MGD = 3,785 m^3/day); and P_D = design population equivalent.

The dependence of construction cost on population equivalent reflects dependence on the amounts of organic matter, (i.e., BOD_5 and TSS) in the wastewater, as opposed to dependence on the design flow alone. Designing the plant to handle a given population equivalent of waste approximates treatment of the wastewater to meet the same effluent concentration standards, regardless of the quantity of flow, within reasonable limits.

The model assumes that the population will grow exponentially at $100k\%/\text{yr}$ and that wastewater generation will remain constant at C gal/cap·day (1 gal/cap·day = 3.785×10^{-3} m^3/cap·day). This is equivalent to assuming that wastewater from industrial/commercial sources will also increase, and that residential sources will continue to account for $100b\%$ of the treatment plant influent. The capacity of the existing wastewater treatment plant, Q_{max}, will be reached in year x_0. Solving Eq. 3 when $Q = Q_{max}$ and $t = x_0$, x_0 can be calculated as:

$$x_0 = \ln \frac{\left[\dfrac{10^6 Q_{max}}{(CP_0)}\right]}{k} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (13)$$

When the capacity of the existing wastewater treatment plant is reached, the model assumes that a second treatment plant will be constructed with sufficient capacity to handle the excess flow due to the increased population and industrial/commercial growth expected throughout the time horizon of the project (i.e., to year T).

With no household water conservation program, the required design flow of the new plant, Q_D, can be calculated from Eq. 14, assuming that the design flow can be estimated as 2.25 times the average daily flow (11):

$$Q_D = 2.25(10^{-6} CP_0 e^{kT} - Q_{max}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

With a household water conservation program, the required design flow for the new plant, Q'_D, is:

$$Q'_D = 2.25(10^{-6} CP_0(1 - abs)\, e^{kT} - Q_{max}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (15)$$

If inflation increases the construction cost at $100\ i_2\%/\text{yr}$ and the discount rate of the municipal wastewater treatment utility is $100r\%/\text{yr}$, the effective interest rate for construction cost, C_r, is:

$$C_r = r - i_2 - ri_2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (16)$$

Assuming continuous compounding, the present equivalent of the construction cost without a household water conservation program, PCOST, is:

$$\text{PCOST} = 52{,}500\, P_D^{0.461} Q_D^{0.429} e^{-C_r x_0} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (17)$$

With household water conservation, capacity of the existing treatment plant is not reached until year x_c, in which:

$$x_c = \ln \frac{\left\{\dfrac{10^6 Q_{max}}{[(1 - abs)\, CP_0]}\right\}}{k} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (18)$$

Construction of the new plant is delayed until this time, so the present equivalent of the construction cost with a household water conservation program, PCOST′, is:

$$\text{PCOST}' = 52{,}500\, P_D^{0.461} Q'^{0.429}_D e^{-C_r x_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (19)$$

Setting the design population equivalents equal for the two plants (i.e., with

and without household water conservation) approximates the increased removal efficiency required for the treatment plant which incorporates household water conservation. The design population equivalent is defined as the population equivalent in excess of that which can be handled by the existing wastewater treatment plant at full capacity. This can be expressed as:

$$P_D = \left(\frac{P_0}{b}\right)(e^{kT} - e^{kx_0}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (20)$$

The net present value of the construction cost savings due to household water conservation measures, CSAV, is:

$$\text{CSAV} = \text{PCOST} - \text{PCOST}' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (21)$$

Cost of Household Water Conservation

To compare the costs of a household water conservation program with the resulting savings, it is necessary to estimate the present equivalent cost of implementing the program. To facilitate this calculation, the model assumes that the wastewater treatment utility pays for the water conservation devices used to achieve the reduction in household wastewater flow, and that the homeowners pay the same monthly charges both for water supply and for wastewater treatment after the program is implemented as before. The homeowner will realize a savings in energy costs due to implementation of devices that reduce hot water usage. However, these savings and any benefits (e.g., longer lasting hot water supply) or disbenefits (e.g., decreased shower pressure) to the household user are neglected for the purpose of this analysis. Placing all costs and savings on the side of the wastewater treatment utility simplifies the cost-effectiveness comparison. This assumption is made to allow comparison of costs and savings and should not be construed as necessarily the optimal method for implementing a water conservation program.

To achieve a constant percentage reduction in wastewater flow over time, it is assumed that the utility buys a sufficient number of devices to serve the entire population expected throughout the project time horizon (up to year T), and that all the devices are bought in the year in which the utility chooses to implement the conservation program. While only $100a\%$ of the households actually implement the conservation measure, conservation devices are purchased by the utility for all the households. This effectively means that $100(1 - a)\%$ of the households receive and discard the conservation devices.

If there are N persons per water-consuming fixture (e.g., toilet or shower), the total number of devices required is:

$$D = \left(\frac{P_0}{N}\right) e^{kT} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (22)$$

in which D = total number of household water conservation devices.

If the cost of conservation devices increases at $100i_3\%/\text{yr}$ due to inflation and the discount rate of the wastewater treatment utility is $100r\%/\text{yr}$, the effective interest rate for household water conservation cost, D_r, is:

$$D_r = r - i_3 - ri_3 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (23)$$

Assuming that the devices currently cost R dollars each; that D devices will be purchased by the wastewater treatment utility in year x_p; and continuous compounding, then the present equivalent cost of the purchase, DCOST, is:

$$\text{DCOST} = DRe^{-D_r x_p} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (24)$$

Eqs. 11, 21, and 24 can be combined to estimate the net present savings from a household water conservation program, TSAV, as:

$$\text{TSAV} = \text{OSAV} + \text{CSAV} - \text{DCOST}$$

Application of Model

Table 4 lists the assumed values for the parameters which were used to evaluate the cost-effectiveness of household water conservation as a component in a

TABLE 4.—Assumed Parameter Values for Base Case Cost-Effectiveness Analysis

Variable (1)	Name (2)	Value (3)
Q_{max}	Existing treatment plant capacity	41.64×10^4 m^3/day (110 MGD)
P_0	Population in year zero	600,000 persons
k	Population growth rate	2%/yr
C	Wastewater generation rate	0.549 m^3/cap·day (145 gal/cap·day)
b	Percent of total influent due to residential sources	45%
i_1	O&M cost inflation rate	5%/yr
i_2	Construction cost inflation rate	6%/yr
i_3	Conservation device inflation rate	5%/yr
T	Project time horizon	30 yr
r	Wastewater treatment utility discount rate	10%/yr
a	Percent of households participating	80%
N	Average number of persons/fixture	2.5 persons/toilet or shower
s	Percent of total household flow reduction due to:	
	toilet dam	10%
	shower flow restrictor	10%
R	Unit cost for:	
	toilet dam	$7.00
	shower flow restrictor	$1.00

hypothetical municipal wastewater treatment strategy. These values are collectively called the "base case." Table 1 shows that bathing and toilet flushing are the two uses which demand the largest amounts of household water. Devices exist to reduce substantially the water used in toilet flushing and showering with little or no perceptible effect on the household user. Table 2 indicates that in combination these two devices (dams and flow restrictors) could reduce total household water use by up to 35%. For this analysis, a more conservative value of 10% reduction due to each of the two devices was used. This accounts

for the variable effectiveness of the devices from one household to another because of variations in water pressure and use habits.

To insure that the prices for the devices used in this analysis are realistic, the assumed prices are based on actual list prices for specific models. Currently, one manufacturer advertises a shower flow restrictor at a unit price of $0.39 (4). To account for shipping and administrative costs, a price of $1.00 per device was assumed. Another manufacturer presently markets a toilet tank dam for a few dollars (13). Since the dams are considerably larger and heavier than the flow restrictors, a larger shipping cost was allotted bringing the assumed price per device to $7.00. These estimates are probably somewhat high, since a utility purchasing the large quantities of conservation devices required for a community-wide conservation program could almost certainly obtain a lower unit price. No installation cost is included since it is assumed that most people will be able to install them by themselves.

By varying the year in which the conservation devices were initially bought and installed, it was found that the optimal year to begin a specific household water conservation measure is dependent on the price of the device. For devices that are relatively inexpensive, it is cost-effective to implement them soon, in order to take advantage of the O&M cost savings, which begins to accrue in the year the flow reduction is made. As the devices become more expensive, it becomes cost-effective to wait, in order to discount the cost of the conservation devices. A device which achieves a 10% reduction in flow is optimally implemented in a household water conservation program in year zero if it costs $6.00 or less; in year 3 if it costs $7.00; in year 10 if it costs $8.00; and in year 12 if it costs $9.00 or more. Without household water conservation, capacity of the existing wastewater treatment plant will be reached in year 12, so that is the latest a conservation program would be implemented. The net savings for the variously-priced devices, implemented in their optimal years, are shown in Table 5.

Since only 45% of the total wastewater flow is residential, the 10% reduction in household flow for 80% of the households amounts to a 3.6% reduction in total wastewater flow. This delays the need for a new plant from year 12–year 14. Combined with the smaller design flow, this delay results in a present equivalent construction cost savings of $10,260,000 regardless of the year in which the 10% reduction in flow is made. The net present savings per capita in Table 5 is calculated by dividing the total net present savings in dollars by the population in year 30, to be consistent with the population that was used to calculate the total number of devices necessary for the program.

Table 5 shows two important results. First, that it may be cost-effective to implement a household water conservation program in more than one stage. If the utility were to use $1.00 flow restrictors and $7.00 toilet dams, total net savings could be maximized by purchasing and installing the flow restrictors immediately (in year zero) and waiting until year 3 to install the toilet tank dams, unless the information and distribution costs from separating the components of the conservation program increased the total cost significantly. The total net present savings for the two devices in conjunction cannot be found by simply adding the separate savings from the two devices in Table 5. Table 6 shows the net present savings which results from implementing several different household water conservation programs.

The second important result shown by Table 5, is that the net present savings for a 10% household wastewater flow reduction (i.e., 3.6% total wastewater

TABLE 5.—Net Present Costs and Savings for Water Conservation Program Which Achieves 10% Reduction in Household Flow

Program cost/household, in dollars (1)	Optimal year for implementation (2)	Net present O&M cost savings, in millions of dollars (3)	Net present conservation costs, in millions of dollars (4)	Net present savings, in millions of dollars (5)	Net present savings per capita, in dollars/person (6)
1.00	0	2.75	0.44	12.58	11.51
2.00	0	2.75	0.87	12.14	11.11
3.00	0	2.75	1.31	11.83	10.82
4.00	0	2.75	1.75	11.27	10.31
5.00	0	2.75	2.19	10.83	9.91
6.00	0	2.75	2.62	10.39	9.51
7.00	3	2.37	2.67	9.97	9.12
8.00	10	1.48	2.21	9.64	8.82
9.00	12	1.39	2.27	9.38	8.58
10.00	12	1.39	2.52	9.13	8.35
15.00	12	1.39	3.78	7.87	7.20
20.00	12	1.39	5.04	6.61	6.05
25.00	12	1.39	6.30	5.36	4.90
30.00	12	1.39	7.55	4.10	3.75
35.00	12	1.39	8.81	2.84	2.60
40.00	12	1.39	10.07	1.48	1.44
45.00	12	1.39	11.33	0.32	0.29
50.00	12	1.39	12.59	−0.94	−0.86

TABLE 6.—Net Present Savings from Alternative Household Water Conservation Programs

Program alternative (1)	Household Water Conservation Measures			Net present savings, in millions of dollars (5)	Net present savings per capita, in dollars/person (6)
	Percent household flow reduction (2)	Year of implementation (3)	Cost/device, in dollars (4)		
I	10	0	$1.00	$12.58	$11.51
II	10	0	$1.00		
	10	3	$7.00	$22.09	$20.21
III	10	0	$1.00		
	10	10	$8.00	$21.77	$19.91

flow reduction) is positive for device costs up to approx $46.00 each. Even with extremely high administration and distribution costs, the cost per device

of the simple conservation equipment under consideration here would almost certainly be well below this.

Sensitivity Analysis

The values of the input parameters assumed for the "base case" are obviously not the only reasonable estimates that could have been selected. Some of the variables, such as the inflation rates, depend on future events and are therefore intrinsically unknowable. Others, such as population growth rate, the fraction of total wastewater flow which is residential, wastewater generation rate, and number of persons per fixture, vary according to the specific character of the community under consideration. To insure that the results of applying the model would not be grossly affected by variations in the parameter values, a sensitivity analysis was performed on some of the more important variables.

The first variable tested for sensitivity is the discount rate of the wastewater treatment utility (r). Fig. 2 shows the results of a household water conservation program that achieves a 10% reduction in household wastewater flow at a cost

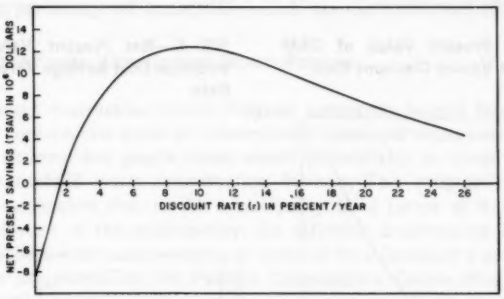

FIG. 2.—Net Present Savings Versus Discount Rate

of $1.00 per device (shower flow restrictors), using a range of values for the discount rate. The graph shows that for discount rates above 2%/yr the net present savings for the program is positive. The shape of the curve is explained by Figs. 3 and 4, which show separately the savings in O&M costs and in construction costs over this same range of discount rates. Since the conservation program is initiated in year zero, which is the optimal year of implementation for a $1.00 device, total costs of conservation are unaffected by the discount rate. The present equivalent of O&M cost savings decreases, as the discount rate increases, because the future savings become less valuable in present equivalent dollars. The construction cost savings, on the other hand, is actually negative for low discount rates, since delaying construction costs which are not heavily discounted does not offset the inflation rate. Thus the smaller treatment plant constructed later is more expensive than the larger one constructed earlier. The construction savings reach a peak at a discount rate of approx 12%/yr and decline because the future construction savings themselves are highly discounted.

Figs. 3 and 4 show that the construction cost savings for the "base case" is about 5 times the O&M cost savings. It is reasonable to ask, therefore, whether household water conservation would still be a cost-effective option in a stable community, in which the capacity of the existing wastewater treatment plant is never exceeded, and no construction takes place. Fig. 5 shows the results of varying the population growth rate (k). Even in a no-growth community,

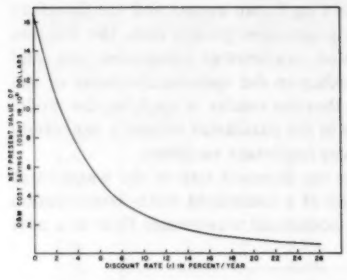

FIG. 3.—Net Present Value of O&M Cost Savings Versus Discount Rate

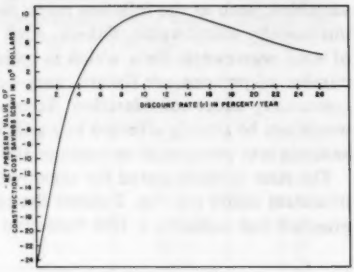

FIG. 4.—Net Present Value of Construction Cost Savings Versus Discount Rate

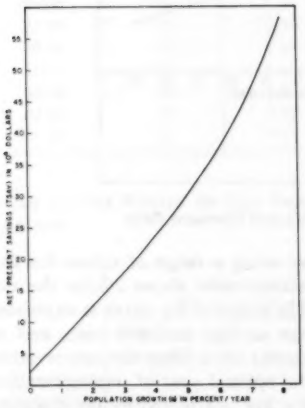

FIG. 5.—Net Present Savings Versus Population Growth Rate

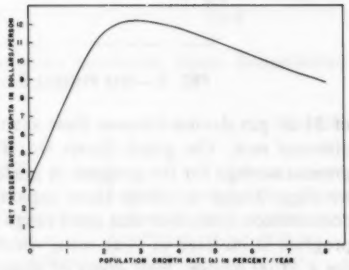

FIG. 6.—Net Present Savings Per Capita Versus Population Growth Rate

where construction cost savings is zero, the net present savings is still positive, although it is reduced to $1,920,000 from $12,580,000 at the 2%/yr population growth rate in the "base case." Higher population growth rates result in greater net present savings, but, as shown in Fig. 6, the net present savings per capita peaks at approximately 3%/yr. Above this point, net present savings per capita decreases because the savings is spread out over a larger number of people.

Other factors besides those considered above could alter the results of the "base case." Clearly, increasing the treatment cost inflation rate (i_1) would increase the savings from conservation, while increasing the inflation rates for construction cost and cost of conservation devices (i_2 and i_3) would decrease the savings from conservation. Increasing the number of people per water-consuming fixture (N) would decrease the cost of conservation by requiring fewer total devices to serve the same population, and would thus increase net savings from conservation. Increasing wastewater generation rate (C) implies more room for conservation but, since this parameter also includes industrial/commercial wastewater it also implies a smaller value for b, the fraction of the total wastewater flow that is residential. The smaller the fraction of the total wastewater flow that is residential, the smaller the impact will be from reducing residential flow and thus, the smaller the net savings from conservation. This means that a community which is smaller than the 600,000 population in the "base case," and has a higher percentage of residential wastewater flow than the "base case" assumption of 45%, could expect to reduce total flow by a greater percentage and therefore, obtain larger net present savings per capita. The effectiveness of water conservation could also be increased or decreased by increasing or decreasing the percentage of households which use the conservation devices (a).

Implications for Federal Construction Grants Program

The Federal Construction Grants Program authorizes federal funds to cover 75% of the construction costs of conventional municipal wastewater treatment plants. No construction grants funds would be available to cover the cost of providing household water conservation devices. This program reduces the effective construction cost to the municipality by a factor of 4. Thus, from the point of view of the municipality, the effective construction cost savings due to household water conservation is reduced by a factor of 4 as well. Table 7 illustrates the effects of the Federal Construction Grants Program on the same household water conservation programs shown in Table 6. For the 10% household flow reduction at a cost of $1.00 per device in the "base case,"

TABLE 7.—Net Present Savings from Alternative Household Water Conservation Programs Considering Federal Construction Grants Program

Program alternative (1)	Household Water Conservation Measures			Net present savings, in millions of dollars (5)	Net present savings per capita, in dollars/person (6)
	Percent household flow reduction (2)	Year of implementation (3)	Cost/device, in dollars (4)		
I	10	0	$1.00	$4.88	$4.47
II	10	0	$1.00	$7.04	$6.44
	10	3	$7.00		
III	10	0	$1.00	$6.71	$6.14
	10	10	$8.00		

the construction grants program reduces the net present savings to $4,880,000 or $4.47 per capita, a 61% decrease. The maximum cost per conservation device to achieve a 10% reduction in household wastewater flow and still have a positive net present savings is reduced from $46.00 to $14.00 as a result of the Federal Construction Grants Program.

These results show that, while the Federal Construction Grants Program may not make household water conservation appear financially unattractive to the municipality, when it is actually economically cost-effective, it does act as a disincentive and may spell the difference in marginal cases and in cases where state construction grants further reduce the effective construction cost to the municipality. One method by which the government could encourage use of water conservation where it is cost-effective would be to fund 75% of the cost of conservation as well as 75% of the cost of construction.

Summary and Conclusions

This study has shown that, for a wide range of reasonable assumptions, household water conservation is a cost-effective flow-reduction measure to include in the overall strategy of a wastewater utility. The interacting effects of the time value of money and of inflation were considered in analyzing the consequences of flow reduction: delaying the construction of additional wastewater treatment facilities; reducing the required capacity of the additional facilities; and lowering the operations and maintenance cost of both the existing and the additional facility.

A cost-effectiveness analysis of flow-reduction measures, including household water conservation, is mandated as part of the planning process (Step 1) under the Federal Construction Grants Program. However, construction grants funds may not be used to cover the cost of flow reduction through household water conservation programs. As a result, although explicit regulations exist requiring a municipality to consider reducing the cost of constructing additional wastewater treatment facilities through household water conservation techniques, a municipality would pay at most 25% of the cost of constructing new facilities, but 100% of the cost of a household water conservation program. While this fact would still not make household water conservation financially unattractive from the point of view of a typical municipality, the effective net present savings for the municipality is reduced in the range of 60%–70%. In marginal cases, and in those states which provide additional construction grants funds (further reducing the share of construction costs paid by the municipality) construction grants may act as a disincentive to maximizing the role of household water conservation in municipal wastewater treatment strategies. Although household water conservation may be cost-effective, it may also be financially unattractive from the viewpoint of a municipality which must pay the total costs of a conservation program, but substantially less than 25% of the costs of constructing additional wastewater treatment facilities. In this instance, it is to the municipality's advantage to consider only the minimum level of household water conservation which will satisfy the regulations of the construction grants program. If action is taken to relieve this potential disincentive, it is likely that flow reduction through household water conservation will be both costeffective and financially attractive for a greater number of municipalities and will assume an expanded role in wastewater treatment strategies.

Several improvements and refinements could be made in the model, particularly in the area of the simplified equations used to estimate O&M costs and construction costs. Literature documenting the dependence of these two costs on the required efficiency of the plant and the volume of wastewater treated could not be located for this study. The equations which were used considered population equivalent and volume treated by the plant. Keeping population equivalent constant in the conservation and non-conservation cases was used to approximate the increased treatment plant efficiency required when household water conservation is implemented. Because population equivalent remained constant and volume treated was reduced, the equations could only estimate reductions for both O&M costs and construction costs due to household water conservation. Theoretically, this need not be true. However, given the very small increase in efficiency required, it would seem likely that some decrease in O&M costs and construction costs would result from household water conservation, but perhaps not so great as this analysis forecasts. Additional work is needed in this area of the model. Cost estimates based upon consideration of the individual unit processes are a potential methodological improvement. Furthermore, this analysis assumes that additional wastewater treatment capacity will be provided by constructing a new plant rather than expanding the existing plant. Although equations to estimate the cost of expanding wastewater treatment plants do exist (9) the range of application of the equations is very limited. This represents another area for improvement. In view of the results of this study and the sensitivity analysis, it appears that household water conservation is an important component of municipal wastewater treatment strategies and that additional federal policies to encourage broad application should be considered.

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Real-Time Rainfall Monitoring-Prediction System and Urban Hydrologic Operations

By Floyd A. Huff,[1] John L. Vogel,[2] and Stanley A. Changnon, Jr.[3]

Introduction

During a 4-yr peroid (1975–1979), a comprehensive hydrometeorological research program was carried out in the Chicago metropolitan area. This research was jointly sponsored by the Illinois State Water Survey and the National Science Foundation, and was performed with the close cooperation of city, state, and Federal agencies and private engineering firms having an interest in the results of the research. One of the prime objectives of the research was the development and evaluation of a real-time monitoring-prediction system for facilitating and substantially improving the operation of urban storm-sanitary sewer systems. Chicago was selected as the study site because of its complex water system which must provide 4,600,000 people with fresh water for domestic and industrial uses; maintain water levels for a major shipping canal; operate a combined storm and sanitary sewer system; and, provide water storage to prevent flooding (11,12). This paper is devoted to a brief review of the development program and a summary of the results of the evaluation of the system developed under the 4-yr program.

Development of System

Optimizing operations of urban water control systems requires major improvement in the utilization of meteorological data (3,7). The Chicago project was initiated to help meet this need, since it had become apparent to the writers by 1974 that the meteorological input could be greatly improved through the proper use of combined radar-raingage measurements. Furthermore, it was recognized that this is the only type of system presently available which has the potential for satisfying the hydrologic need to specify with a short lead time such important storm parameters, as movement, intensification, areal extent, surface rainfall rates, and rainfall volume. The development of the system was

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Note.—Discussion open until March 1, 1982. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on January 19, 1981. This paper is part of the Journal of the Water Resources Planning and Management Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. WR2, October, 1981. ISSN 0145-0743/81/0002-0419/$01.00.

made possible by major advances in radar meteorology in recent years which have provided the instrumentation and technology to achieve the project's objective.

Development of the system required an extensive field and analysis program during the first 2.5 yr of the project. The field installation involved a dense recording raingage network (1 gage every 9 mile^2 or 25 km^2), and a sophisticated radar system. The radar system was designated HOT (*H*ydrometeorological *O*perational *T*ool), and is a 10-cm wavelength radar with associated signal processor and minicomputer for rapid digitization of the radar-indicated rainfall information (7). In the Illinois State Water Survey (ISWS) system, real-time raingage data are used to adjust the radar-indicated rainfall rates periodically to overcome inherent errors in the radar's indirect measurement of rainfall rates, which are computed from empirical relations between radar reflectivity and rainfall rate (2). The dense raingage network was used to determine the raingage density required for maintaining accurate spatial and temporal calibration of the radar-indicated rainfall distribution.

Analyses were performed concurrently with the field operations to expedite development of the system. Basic tasks included: (1) Designation of the most appropriate technique for adjustment of the radar-indicated rainfall field; and (2) development of various software packages needed by the onsite minicomputer for the real-time monitoring and predicting of storms approaching and crossing the urban area. After evaluating various adjustment procedures, the Brandes technique (1), with certain modifications required for real-time operations, was selected for use in the ISWS system. With this technique, adjustment of the radar-rainfall field is made at short intervals throughout each storm using telemetered raingage data (10). Forecasts of quantitative rainfall requires that real-time echo tracking and analysis programs be written. The programs developed by Wiggert, et al. (13), for the Florida Area Cumulus Experiment (FACE), were adopted and modified to make them applicable to a real-time operation over a 16,384 mile^2 (42,435 km^2) region or an area 128 mile (206 km) on a side. These programs calculated various echo characteristics (location, size, rain rate, rain volume, movement) at short intervals during each storm. This data were then fed into programs which advected the rain system across the city and made a quantitative rainfall forecast (2).

Components of the systems were defined, the necessary software assembled, raingaging requirements for the real-time operations specified, and operational procedures determined during system development period (2,8). Testing and evaluation of the system were carried out in an 18-month period beginning in September, 1978. Evaluation concentrated on a 2-month demonstration period from mid-June–mid-August 1979, when the system was tested under real-time operational conditions in cooperation with the Metropolitan Sanitary District (MSD) of Greater Chicago, who operate the complex storm-sanitary sewer system in the metropolitan area.

Monitoring and Prediction During Demonstration

Introduction.—The basic elements of the monitoring-prediction system were: (1) A dedicated 10-cm radar equipped with digital processor and minicomputer; (2) real-time rainfall data supplied by 22 telemetered raingages operated by MSD

in the metropolitan area; (3) a communications link between the radar operational center (HOT in Fig. 1) and the MSD Operational Center to obtain telemetered rainfall data from MSD and transmit real-time monitoring and prediction information to MSD; and (4) current weather information from the National Weather Service for preparing alerts of potentially heavy rain events 6 hr or longer in advance of arrival and for scheduling operations. The location of the MSD raingages and the three urban sections (north, central, south) for which predictions were issued are shown in Fig. 1. Fig. 2 shows the dense network of 71 recording raingages in the metropolitan area; this network was used as a standard of comparison for evaluating the rainfall monitoring and predicting under real-time operational conditions.

The ISWS rainfall monitoring-prediction system employed a man-machine mix during the demonstration period. The modified FACE tracking system requires

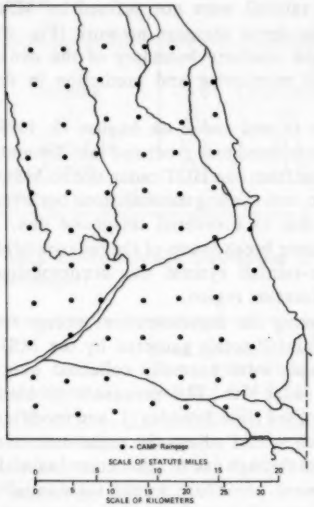

FIG. 1.—NSD Network of Telemetered Raingages

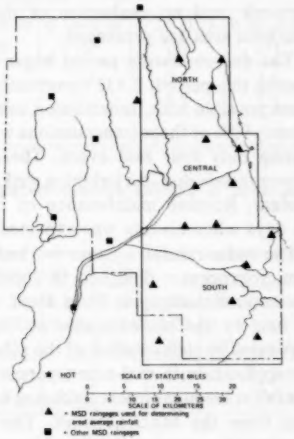

FIG. 2.—Dense Network used for Evaluation of Monitoring-Prediction System in 1979

echo computations at intervals of 5 min or less for accurate tracking and definition of echo characteristics. Because of limitations of our onsite minicomputer, automatic 5-min tracking could not be done, so computations were made at 10-min intervals with a skilled operator providing the ability to match echoes from one 10-min interval to the next. The computer isolated the echoes, did the bookkeeping, and made computations. Monitoring and prediction information was transmitted to MSD from the HOT radar site every 30 min from the time the radar detected a storm approaching or forming over the urban area. Once a rain area was in range of the radar, objective schemes monitored radar-indicated

rainfall amounts on a grid of 2 miles (3.2 km) or an equivalent of one raingage every 2 miles (3.2 km) over 16,384 mile^2 (42,435 km^2).

At each 30-min interval, the rainfall that occurred in the last 30 min over each of the three urban sections (Fig. 1) and the total amount (accumulated rainfall) since the start of the storm were transmitted. At the same time, the amount predicted to fall in the next 30 min, 60 min, and 120 min in each section and the storm total (measured plus predicted) expected at the end of each of these three prediction intervals were furnished to MSD. Updating of the monitoring and prediction continued at 30-min intervals until the storm departed the urban area.

The combined radar-raingage system was designed to monitor and predict quantitative precipitation amounts, and therefore, separate evaluations were made of these two functions. The primary evaluation tool for both was the dense recording raingage network (Fig. 2). The three areas (north, central, south in Fig. 1) for monitoring and forecasting of rainfall were not defined by MSD until May 1979, after the installation of the dense raingage network (Fig. 2). The southern area extended well beyond the southern boundary of the dense network, and no evaluation of the rainfall monitoring and prediction in the southern area are presented.

The demonstration period began on June 18 and ended on August 15, 1979. During this period, 2,832 transmissions of monitored and predicted rain amounts were possible with transmission every 30 min from the HOT radar site to MSD. Nearly 99% of these transmissions were made, and missing transmissions occurred during only four rain event. These were due to a severed telephone line, a power failure due to a lightning strike, and minor breakdowns of the radar-rainfall system. Routine maintenance of the radar-rainfall system was accomplished on days when no rain was expected in the forecast region.

The radar-rainfall system worked well during the demonstration, except for a microprocessor designed to accumulate rainfall totals gathered by the MSD telemetered raingages. Data from these gages were normally collected every 30 min by the mincomputer at the HOT radar site. The processor problem prevented implementation of the scheme, adopted from Brandes (1) and modified for application in real-time operations, which would adjust the radar-indicated rainfall at frequent intervals during each storm through use of telemetered rainfall data from the MSD network. This adjustment procedure would substantially improve the accuracy of the radar-measured rainfall rates for reasons previously mentioned, and, thereby, make the system more useful in urban hydrologic operations. Part of the microprocessor problems were corrected by late July, and data from some of the MSD gages were received satisfactorily by July 31, so that this real-time raingage data could be used by the operator to improve the accuracy of the radar-rainfall measurements during the period from August 1–August 15, and, thus, improve the quality of the monitoring-prediction activities to some degree.

Method of Evaluation.—As a result of the foregoing problems, the verification of the monitored and predicted rainfall amounts was divided into two groups. The first included all storms from the beginning of the demonstration period to July 31. This period is referred to as "June–July." The second period began on August 1 and ended August 15, and is referred to as "August."

The monitored and predicted rainfall values provided to MSD included areal

average rainfall amounts for the northern, central, and southern basins delineated by MSD (Fig. 1). A storm was defined as any rain period separated by more than 3 hr without rain over the monitoring-prediction area. The evaluation of these monitored and forecasted amounts was done only for those periods when rain threatened or occurred over one of the three regions. Thus, all zero monitored amounts or forecasts of zero rainfall were eliminated, even though they were *all* correct. Their inclusion would greatly improve the accuracy values since they represent over 75% of the possible values. The forecasts were evaluated from either two hours before a rainfall event began or from the time the first forecast indicated measurable rainfall in one of the three urban regions, whichever came first. The evaluation of the monitored rain amounts began from the time an areal rainfall amount of at least 0.01 in. (0.25 mm) or more was measured

TABLE 1.—Percentage Distribution of 30-min Average Rainfall in June–July and August

Months (1)	Rainfall Amounts, in inches					
	<0.01 (2)	0.01–0.04 (3)	0.05–0.09 (4)	0.10–0.19 (5)	0.20–0.29 (6)	0.30–0.99 (7)
June–July	65.4	22.5	6.8	3.2	0.5	1.6
August	57.4	21.5	10.3	7.0	2.5	1.3

Note: 1 in. = 25.4 mm.

TABLE 2.—Percentage Distribution of Accumulated Average Rainfall in June–July and August

Months (1)	Rainfall Amounts, in inches									
	<0.01 (2)	0.01–0.04 (3)	0.05–0.09 (4)	0.10–0.19 (5)	0.20–0.29 (6)	0.30–0.49 (7)	0.50–0.74 (8)	0.75–0.99 (9)	1.00–1.49 (10)	≥1.5 (11)
June–July	40.1	21.6	5.6	11.5	6.3	7.7	3.2	1.1	2.9	0.0
August	31.8	16.5	5.0	9.1	7.0	5.8	9.9	12.0	1.7	1.2

Note: 1 in. = 25.4 mm.

by the raingages or the radar indicated an areal average of 0.01 in. (0.25 mm) or more in any of the three areas.

The evaluation of both the monitored and predicted values was done by calculating the difference between the monitored or predicted rainfall and the rainfall measured by the dense raingage network. The dense network measurements of areal mean rainfall were assumed to be the "true" rainfall; thus, the difference between the network and the monitored or predicted rainfall was defined as the monitoring or forecasting error.

Rainfall Distribution.—The rainfall from the dense raingage network collected during the demonstration period was later analyzed to determine the distribution of 30-min and accumulated rainfall amounts for guidance in subsequent analyses and interpretation of results. Since the verification period was divided into

June–July and August, the rainfall analysis was divided similarly. Accumulated and 30-min averages of rain were obtained for each 30-min period in which rain fell in at least one raingage, and the averaging continued throughout the storm. During the demonstration period, 343 periods of 30-min and accumulated rain were averaged.

More than 50% of the 30-min rain periods in June–July and August averaged less than 0.01 in. (0.25 mm) of rain (Table 1). Less than 11% had amounts equalling or exceeding 0.10 in. (2.5 mm), and these are the only intensities that are usually important to urban hydrologists during heavy rainstorm operations. Less than 2% of the 30-min periods had moderate intensities exceeding 0.30 in. (7.6 mm). None exceeded 1.00 in. (25.4 mm) in 30 min. Thus, analyses and conclusions had to be based upon a very limited sample of hydrologically-important rain intensities, due to the short demonstration period and relatively few natural occurrences of heavy storm rainfalls.

The distribution of accumulated rainfall amounts as a percentage of the total observations is shown in Table 2. Accumulated amounts are the total rainfall amounts for the storm at the end of each individual 30-min period. Table 2 shows that rainstorms tended to be more intense in August. For example, approx 47% of the accumulations in August equalled or exceeded 0.10 in. (2.5 mm) compared to 33% in June–July. Similarly, approx 25% of the August accumulations equalled or exceeded 0.50 in. (12.7 mm) compared to 7% in June–July. The average rain for each storm during June–July was 0.17 in. (4.3 mm), compared to 0.31 in. (7.9 mm) in August. Overall, the rainfall analysis indicated that many light scattered rainstorms occurred during June–July intermingled with heavier, more general rains. The August storms were characterized by heavier, widespread rainfall.

Verification of Monitoring

During the demonstration period, monitored values of quantitative rainfall amounts were obtained for 30-min periods and for the accumulated rainfall during each storm. Quantitative amounts were obtained from: (1) The dense raingage network which encompassed the northern and central sections (Fig. 2); (2) the MSD telemetered raingage values (Fig. 1); (3) the radar-indicated rainfall; and (4) the man-machine estimates of rainfall. The radar-indicated rainfall amounts were calculated using a radar reflectivity-rainfall rate relation derived as part of the monitoring-prediction studies. The standard of comparison was rainfall amounts measured by the dense network. Based on extensive raingaging studies by the Survey, which has operated dense networks in Illinois for the past 30 yr, the Chicago network with gages spaced approx 3 miles (4.8 km) apart should provide accurate measurements of the "true" mean rainfall, areal extent of rainfall and other rainfall parameters for sampling periods of 30-min or longer used in the Chicago studies (6). In the following paragraphs, a brief summary of the highlights of these analyses is provided. The reader is referred to Changnon, et al. (2) for more detailed data and information.

30-Min Monitored Rainfall.—Initially, an analysis was performed to determine the accuracy of monitored rainfall amounts available from the MSD telemetered network. This information is useful in establishing telemetering needs for urban operations, since the MSD network with gages spaced about 6 miles (9.7 km)

apart is a good example of a relatively dense urban network. Furthermore, evaluation of the MSD network is helpful in evaluating the applicability of radar and man-machine combinations for monitoring and prediction in urban real-time operations.

Table 3 provides a brief summary of the MSD network evaluation. All data have been combined to provide a frequency distribution of measurement errors in the north and central sections during June–August 1979. Values are accumulated from left to right in Tables 3–8. Frequency of errors of various magnitude are shown for several groups or classes of areal mean rainfall for 30-min periods within storms. For example, with the areal means of 0.10–0.19 in. (2.5 mm–4.8 mm), there were 31 samples and the MSD measurement errors ranged from 0 in.–0.24 in. (6.4 mm). Of the 31 cases, 15 (48%) were 0.04 in. (1.0 mm) or less, and 24 (77%) had errors of 0.10 in. (2.5 mm) or less.

Inspection of Table 3 shows that if a very high degree of accuracy in mean rainfall measurement is required for short intervals, such as 30 min, the MSD network is probably not adequate. However, for real-time hydrologic operations in which an error of the order of 0.10 in. (2.5 mm) is usually not critical,

TABLE 3.—Frequency Distribution of *MSD Network* Errors in Monitoring 30-min Rainfall during June–August in North and Central Sections

Computed mean, in inches (1)	Number of Cases Enveloped for Given Error, in inches					
	0 (2)	≤0.04 (3)	≤0.10 (4)	≤0.24 (5)	≤0.49 (6)	≤0.99 (7)
0.01–0.04	32	121	123	125	125	126
0.05–0.09	8	41	53			
0.10–0.19	2	15	24	31		
0.20–0.29	0	2	5	6	8	
≥0.30	0	1	4	6		

Note: 1 in. = 25.4 mm.

the network can provide useful telemetered data for real-time operations. Our studies indicate that this error distribution can be improved upon by use of 10-cm radar in conjunction with adequate telemetered gages to adjust the radar-indicated rainfall for various types of errors that are inherent to such systems in the quantitative measurement of rainfall.

As indicated earlier, a microprocessor problem prevented adjustment of the radar-indicated rainfall rates with telemetered raingage data through use of the modified Brandes technique developed as part of this research. Table 4 shows the frequency distribution of errors with the unadjusted radar observations of the 10-cm, HOT radar. The contents of the table are the same as Table 3.

For very light rainfalls (0.01 in.–0.04 in. or 0.25 mm–1.0 mm), the radar accuracy is approximately equivalent to the MSD network, which is a relatively dense urban network. However, for moderate 30-min rainfall (0.10 in.–0.19 in. or 2.5 mm–4.8 mm), the radar accuracy is less than that of the MSD network. Thus, only 24% of the radar errors for areal means of 0.10 in.–0.19 in. (2.5 mm–4.8 mm) were 0.04 in. (1.0 mm) or less, compared to 48% for the MSD network. With areal means of 0.30 in. (7.6 mm) or more 43% (9) of the radar

cases had errors in the range from 0.25 in.–0.49 in. (6.4 mm–12.5 mm). The majority of these exceeded 100%.

The distribution of errors with the unadjusted radar measurements in Table 4 shows the need for an adjustment procedure to maximize the benefits of radar in real-time, prediction-monitoring of heavy rainstorms. This has been stressed in our earlier research reports (8,9) and by others (4,5,14).

At the present stage of technological development, it was concluded earlier that a combination of man and machines was needed to optimize the real-time, monitoring-prediction system developed for the Chicago urban area. This requires an experienced radar meteorologist to examine, evaluate, and adjust when

TABLE 4.—Frequency Distribution of *Unadjusted Radar Measurement* Errors in Monitoring 30-min Rainfall during June–August in North and Central Sections

Computed mean, in inches (1)	Number of Cases Enveloped for Given Error, in inches					
	0 (2)	≤0.04 (3)	≤0.10 (4)	≤0.24 (5)	≤0.49 (6)	≤0.99 (7)
0.01–0.04	17	216	217			
0.05–0.09	0	32	56			
0.10–0.19	0	9	30	37	37	38
0.20–0.29	0	1	5	25		
≥0.30	0	1	5	12	21	

Note: 1 in. = 25.4 mm.

TABLE 5.—Frequency Distribution of *Man-Machine* Errors in Monitoring 30-min Rainfall during June–July in North and Central Sections

Computed mean, in inches (1)	Number of Cases Enveloped for Given Error, in inches				
	≤0.04 (2)	≤0.10 (3)	≤0.24 (4)	≤0.49 (5)	≤0.99 (6)
0.05–0.09	8	13			
0.10–0.19	12	46	47	47	48
0.20–0.29	2	8	16		
≥0.30	0	3	10	13	14

Note: 1 in. = 25.4 mm.

necessary, the computer output pertaining to movement, intensity, rainfall volume and other pertinent factors derived from radar observations of the storm echo field.

The frequency distribution of errors with the man-machine combination used during June-August 1979 is summarized in Tables 5. The June–July and August distributions have been separated, because some telemetered raingage data was available to the duty meteorologist for adjusting the radar echo intensities during August. The August sample was much smaller than that for June–July, so only limited comparison can be made between the two periods.

Inspection of Table 5 indicates (as expected) that the errors become smaller as telemetered data become available for adjusting the radar-indicated rainfall.

Thus, for computed means of 0.10 in.–0.19 in. (2.5 mm–4.8 mm), the error was equal to or less than 0.04 in. (1.0 mm) in 69% (18/26) of the cases in August compared with 25% (12/48) in June–July, when no telemetered raingage data were available. Similarly, 75% (6/8) of the errors for means of 0.20 in.–0.29 in. (5.9 mm–7.4 mm) were equal to or less than 0.10 in. (2.5 mm) in August compared to 50% (8/16) in June–July.

Comparison of the man-machine errors in Table 5 with those for unadjusted radar in Table 4 provides evidence of the superiority of the man-machine combination. For example, combining June–July and August in Table 5, the error in computed means of 0.10 in.–0.19 in. (2.5 mm–4.8 mm) was equal to or less than 0.04 in. (1.0 mm) in 41% (30/74) of the cases for the man-machine method compared with 24% (9/38) for the unadjusted radar estimate. Similarly, the error was 0.10 in. (2.5 mm) or less in 95% (70/74) of the cases for the man-machine mix compared with 79% (30/38) with the unadjusted radar. The difference became more pronounced in the heavier rain intensities. Thus, for computed means of 0.20 in.–0.29 in. (5.0 mm–7.4 mm), the man-machine error was equal to or less than 0.10 in. (2.5 mm) in 58% (14/24) of the man-machine cases compared with 20% (5/25) for the unadjusted radar computations. As

TABLE 6.—Frequency Distribution of *Man-Machine* Errors in Monitoring 30-min Rainfall during August in North and Central Sections

Computed mean, in inches (1)	Number of Cases Enveloped for Given Error, in inches				
	≤0.04 (2)	≤0.10 (3)	≤0.24 (4)	≤0.49 (5)	≤0.99 (6)
0.10–0.19	18	24	26		
0.20–0.29	1	6	8		
≥0.30	0	2	3	3	5

Note: 1 in. = 25.4 mm.

pointed out earlier, the man-machine method was operating at less than maximum effectiveness during most of the demonstration period because telemetered raingage data were not available for guidance in the man-initiated adjustments of machine-computed rainfall intensities. Otherwise, the superiority of the man-machine mix in monitoring rainstorms would very likely have been even more pronounced.

Comparison of Table 5 with Table 3 indicates that the man-machine method of monitoring was equivalent, if not slightly superior, to the relatively dense urban network operated by MSD. For computed means of 0.05 in.–0.09 in. (1.3 mm–2.3 mm), the man-machine error never exceeded 0.10 in. (2.5 mm). For means of 0.10 in.–0.19 in. (2.5 mm–4.8 mm), the man-machine error was equal to or less than 0.10 in. (2.5 mm) in 95% (70/74) of the cases compared with 77% (24/31) for the MSD network. When computed means were 0.20 in.–0.29 in. (5.0 mm–7.4 mm), the man-machine method had errors equal to or less than 0.10 in. (2.5 mm) in 58% (14/24) of the samples compared to 62% (5/8) for the MSD network. With complete telemetered raingage data available to the duty meteorologist, it is anticipated that the man-machine mix

would have exhibited a significant superiority.

Accumulated Rainfall Monitoring.—The summary analyses presented in Tables 3–5 for 30-min rainfall amounts were repeated for the accumulated rainfall amounts. As with the 30-min amounts, detailed analyses are presented in Changnon, et al. (2).

Table 6 shows the frequency distribution of MSD network errors in monitoring accumulated rainfall amounts at 30-min intervals throughout each storm. The

TABLE 7.—Frequency Distribution of *Unadjusted Radar Measurement* Errors in Monitoring Accumulated Rainfall during June–August in North and Central Sections

Computed mean, in inches (1)	Number of Cases Enveloped for Given Error, in inches						
	0 (2)	≤0.04 (3)	≤0.10 (4)	≤0.24 (5)	≤0.49 (6)	≤0.99 (7)	≤1.50 (8)
0.01–0.04	5	167	170	170	178	184	
0.05–0.09	0	15	77				
0.10–0.19	0	3	18	56	64		
0.20–0.29	0	1	5	53	57		
0.30–0.49	0	0	0	29	38		
0.50–0.99	0	2	5	12	46	75	
≥1.00	0	0	0	6	22	83	98

Note: 1 in. = 25.4 mm.

TABLE 8.—Frequency Distribution of *Man-Machine* Errors in Monitoring Accumulated Rainfall during June–July and August in North and Central Sections

Computed mean, in inches (1)	Number of Cases Enveloped for Given Error, in inches													
	0		≤0.04		≤0.10		≤0.24		≤0.49		≤0.99		≤1.50	
	J–J[a] (2)	A[b] (3)	J–J (4)	A (5)	J–J (6)	A (7)	J–J (8)	A (9)	J–J (10)	A (11)	J–J (12)	A (13)	J–J (14)	A (15)
0.05–0.09	2		2		2		2		4					
0.10–0.19	21	12	21	12	82	14	82	14	86	14				
0.20–0.29	5	2	5	2	13	19	31	19	32	19				
0.30–0.49	1	8	1	8	9	9	14	12	18	13	21	13		
0.50–0.99	2	18	2	18	4	28	16	37	30	56	48	57	48	57
≥1.00	0	0	0	0	3	0	7	11	10	17	35	17	49	17

Note: 1 in. = 25.4 mm.
[a]J–J = June–July.
[b]A = August.

same general trends, and, therefore, the same conclusions are reached as pointed out in the discussion of the 30-min amounts summarized in Table 3. Note in Table 6 that the percentage error tends to decrease with increasing rainfall, which is pertinent to urban hydrologic operations. For example, for accumulations of 1.00 in. (25.4 mm) or more, the error never exceeded 0.24 in. (6.1 mm) (less than 25%), whereas maximum errors of the same magnitude occurred with all the lighter amounts.

Table 7 shows the same trends as Table 4 with unadjusted radar measurement of rainfall amounts. That is, radar is approximately equivalent to the MSD network in the measurement of very light amounts, but the network is significantly superior in measuring moderate to heavy rain amounts.

Examination of Table 8 supports the conclusion obtained from Table 5 regarding the significant improvement in the man-machine measurements expected when telemetered raingage data are available to assist in adjustment of the radar-indicated rain rates. Thus, in August when some telemetered data were available to the duty meteorologist, accumulative amounts were measured with smaller errors than in June–July, and the improvement was most pronounced in the heavier rainfalls, which are of primary interest to the urban hydrologist. For example, when computed means were 0.50 in.–0.99 in. (12.7 mm–25.2 mm), 49% (28/57) of the August means had errors of 0.10 in. (2.5 mm) or less, whereas only 8% (4/48) were in this range in June–July storms. With accumulated amounts of 1.00 in. (25.4 mm) or more, all August errors were less than 0.49 in. (12.5 mm), but in June–July only 20% (10/49) were less than 0.49 in. (12.5 mm).

Comparison of Tables 7 and 8 provides additional support for the earlier conclusion regarding the definite superiority of the man-machine monitoring over unadjusted radar measurements of surface rainfall. As one example, for accumulated amounts (computed areal means) in the range from 0.50 in.–0.99 in. (12.7 mm–25.2 mm) only 16% (12/75) of the radar measurements had errors equal to or less than 0.24 in. (6.1 mm), compared with 33% (16/48) in June–July for the man-machine mix with no telemetered raingage data available to the duty meteorologist, and 65% (37/57) in August when part of the telemetered data became available for guidance in adjusting the radar-indicated rain amounts. Comparison of Table 6 and 8 indicates that the MSD network errors in measuring accumulated rainfall amounts tended to be smaller than the man-machine combination in June–July, but about equivalent in August when some, but not all, of the telemetered data became available to the duty meteorologist.

Forecast Verification

Introduction.—During the demonstration period types of two forecasts were made. The first predicted the amount of additional rain expected over the urban areas in the next 30 min, 60 min, and 120 min during a storm. The second gave the anticipated accumulated rain amounts from the beginning of the storm to the end of the next 30 min, 60 min, and 120 min. Verification for both types of forecasts began from the time of the first nonzero forecast for one or more of the three areas, or two hours before an areal average of 0.01 in. (0.25 mm) occurred in any of the areas.

During each storm two forecasting methods were used. The first involved a *semiobjective* forecast that utilized the echo tracking system. The radar meteorologist first corrected computer-derived storm motions which were inconsistent with the expected movement of the storm (based on synoptic weather analyses); adjusted the rate of change of rain intensity or the areal size, or both, projected over the next 120 min; and then placed upper limits on the size and rain intensity of each cell. These changes were made on the basis of continuity of the rain system, experience, and knowledge of synoptic

conditions. The adjustments (if any) were inserted into the minicomputer and were used to calculate the rainfall forecasts for the upcoming 30 min, 60 min, and 120 min.

The second method utilized the semi-objective forecasts modified by the radar meteorologist, if deemed necessary, through use of real-time rain information and knowledge of present weather conditions. These forecasts were transmitted to MSD, and are referred to hereafter as *man-machine* forecasts.

The individual and accumulated semi-objective forecasts were made to the nearest 0.01 in. (0.25 mm) for each of the three urban areas for 30 min, 60 min, and 120 min. However, the man-machine forecasts, transmitted to MSD by the operator, were given to the nearest 0.1 in. (2.5 mm) but the evaluations of both the semi-objective and man-machine were made to the nearest 0.01 in. (0.25 mm).

For the "semi-objective forecasts," rainfall amounts in the urban area were determined solely from the radar-indicated rainfall amounts. Thus, if the radar-indicated amounts were either high or low, the individual or accumulated (or both) rainfall amounts for forecasts in the next 30 min, 60 min, and 120 min would also be either high or low when compared to the dense network measurements used as a comparison standard. However, the "man-machine forecasts" issued by the operator were altered on the basis of the radar-indicated rainfall, the telemetered raingage amounts (when available), and by knowledge of storm characteristics.

The results from the northern and central regions were similar, and have been combined for presentation in this paper. Various types of analyses were made to evaluate the accuracy of the man-machine and semi-objective forecasts (2). These included: (1) Determination of the forecast errors in absolute magnitude and percentages for various intensities of 30-min, 60-min, and 120-min rainfalls, and for accumulated rainfall amounts during the progress of storms; (2) the frequency of overestimates and underestimates of actual rainfall amounts; (3) differences in forecasting accuracy between June–July and August, when some telemetered real-time raingage data became available for guidance; and (4) comparative accuracy of the semi-objective and man-machine forecasts. In the following paragraphs, only a brief summary of the major findings from the various analyses is presented.

Forecasts of Individual 30-Min–120-Min Amounts.—In view of the relatively short sampling period, median forecasting errors were considered the best statistic to provide an overall measure of forecasting accuracy achieved during the demonstration period. These have been used to a large extent to summarize project findings in the ensuing paragraphs.

Table 9 summarizes a comparison between the median forecasting errors for the man-machine and semi-objective techniques. These are shown for intrastorm forecasts of 30-min, 60-min, and 120-min rainfall amounts transmitted to MSD at 30-min intervals throughout each storm. The distribution of errors is shown for various classes of forecast amounts (rainfall intensities). Results have been separated into June–July and August time periods, so as to evaluate the effect of the limited real-time raingage data that became available in August to serve as an additional forecasting guide. The number of cases in each forecast category is also indicated in Table 9. The semi-objective method has a greater number of forecasts because radar, frequently, indicated small rain amounts due to

light rain aloft that evaporated before reaching the ground, or from the detection of large cloud droplets in non-precipitating clouds. The man-machine technique discerned these instances and correctly made zero (no rain) forecast which are not included in the summarized statistics.

Table 9 shows a distinct trend for the man-machine forecasts to be superior to the semi-objective technique. For example, during June–July, the man-machine median error for 30-min forecasts in the range from 0.30 in.–0.99 in. (7.6 mm–25.2 mm) was 16% better (0.04/0.25). For 60-min forecasts, the man-machine error was 9% better (0.03/0.32), and for 120-min forecasts, the improvement was 32% (0.12/0.38). The improvement was greatest for the heavier rainfall amounts, those of most interest to the urban hydrologist. The results support our initial concept that substantial improvement in forecasting accuracy could be achieved with a man-machine mix as opposed to purely objective methods, considering the present level of technological development in the radar-rainfall field.

The August samples were much smaller than those for June–July. Consequently, a thorough evaluation cannot be made of the effect of the August availability of limited telemetered raingage data to the duty meteorologist. Inspection of

TABLE 9.—Frequency Distribution of *Man-Machine* Errors in Monitoring Accumulated Rainfall during June–July in North and Central Sections

Computed mean, in inches (1)	Number of Cases Enveloped for Given Error, in inches						
	0 (2)	≤0.04 (3)	≤0.10 (4)	≤0.24 (5)	≤0.49 (6)	≤0.99 (7)	≤1.50 (8)
0.05–0.09	2	2	2	2	4		
0.10–0.19	21	21	82	82	86		
0.20–0.29	5	5	13	31	32		
0.30–0.49	1	1	9	14	18	21	
0.50–0.99	2	2	4	16	30	48	
≥1.00	0	0	3	7	10	35	49

Note: 1 in. = 25.4 mm.

Table 9 shows a general improvement (smaller median errors) from June–July to August in the man-machine forecasts. However, improvement is also indicated in the semi-objective forecasts which were unaffected by the telemetered raingage data.

Thus, Table 9 suggests that the improvement may only indicate a sampling variation resulting from differences in the type and distribution characteristics of rainfall during the two periods. However, other analyses (2) indicated that the improvement from June–July to August may have been somewhat better for the man-machine forecasts. The man-machine forecasts tended to have a greater percentage of errors < 50% in August than the semi-objective technique. Also, analyses of the percentage of forecasts errors exceeding 0.1 in. (2.5 mm) indicated a greater improvement in August compared to June–July in suppressing large errors by the man-machine methods.

As expected, Table 9 indicates that the magnitude of the forecasting error increases as forecasts are made for increasingly heavy rainfalls. For example, the median error of 60-min forecasts for June–July with the man-machine method

increased gradually from 0.03 in. (0.8 mm) for predictions of 0.10 in.–0.04 in. (0.3 mm–1.0 mm)–0.29 in. (7.4 mm) for forecasts in the range of 0.30 in.–0.99 in. (7.6 mm–25.3 mm). However, the percentage error did not gradually increase with increasing rainfall; in fact, there was a tendency in the man-machine forecasts for it to decrease slightly as the forecast amounts became larger. More detail on the breakdown of both absolute and percentage errors is provided by Changnon, et al. (2).

Forecasts of Accumulated Rainfall Amounts.—Analyses discussed in previous paragraphs were repeated for accumulated rainfall forecasts. Total storm accumulations were predicted every 30 min for the next 30 min, 60 min, and 120 min. That is, amounts expected in these upcoming periods were added to the monitored rainfall totals up to the time of the latest forecast. Results are presented in Table 10. This table shows a pronounced trend for the man-machine forecasts to be more accurate than the semi-objective predictions. In general, this is evident for all forecasting categories and time periods.

Table 10 indicates a substantially stronger trend for an improvement in man-machine forecasts from June–July to August than was apparent for the

TABLE 10.—Frequency Distribution of *Man-Machine* Errors in Monitoring Accumulated Rainfall during August in North and Central Sections

Computed mean, in inches (1)	Number of Cases Enveloped for Given Error, in inches						
	0 (2)	≤0.04 (3)	≤0.10 (4)	≤0.24 (5)	≤0.49 (6)	≤0.99 (7)	≤1.50 (8)
0.10–0.19	12	12	14				
0.20–0.29	2	2	19				
0.30–0.49	8	8	9	12	13		
0.50–0.99	18	18	28	37	56	57	
≥1.00	0	0	0	11	17		

Note: 1 in. = 25.4 mm.

individual 30-min–120-min predictions. For example, in the accumulated rainfall forecasts for 60 min in advance, the median error decreased 0.02 in. (0.5 mm) for forecasts of 0.10 in.–0.19 in. and 0.60 in. (15.2 mm) for predictions in the range from 1.00 in.–1.49 in. (25.4 mm–37.9 mm). Furthermore, the man-machine forecast improvement was substantially greater than the semi-objective change which was erratic with respect to forecast amount and forecast period, and was only slight overall. Thus, the accumulated rainfall analyses provide some solid support for an improvement in the man-machine forecasts with only limited telemetered gage data available to help adjust the machine forecasts.

Similar to the individual period forecasts, the absolute error increased with increasing rainfall, but the percentage error did not change much and tended to decrease slightly with the heavier predicted amounts. In general, the analyses of accumulated rainfall forecasts provide strong support for findings discussed earlier concerning forecasts of individual 30-min, 60-min, and 120-min rainfall amounts.

Summary

A major objective of the 4-yr of research in the Chicago region was to expand

the use of meteorological information in the real-time operation of urban hydrologic systems, so as to improve protection of people, property, and the environment during moderate to heavy rains including potential flash flood situations. Carrying out this objective involved the development of a real-time monitoring-prediction system utilizing a combination of weather radar and telemetered recording raingage data. *Monitoring* of rainfall from storms approaching and crossing the urban area, and *predicting* the rainfall distribution over the urban area for the upcoming 30-min, 60-min, and 120-min were the two basic goals of this research. Such a system could provide data necessary for optimizing the operation of urban hydrologic systems.

During the demonstration period, the rain monitoring accuracy with the man-machine combination was definitely superior to that obtained with unadjusted radar-rainfall measurements, and was equivalent, if not slightly superior, to the measurements made by the relatively dense network of recording raingages operated by MSD in the urban area. For 30-min rain amounts of 0.1 in. (2.5 mm) or greater, the more hydrologically significant events, the man-machine combination had an error of 0.1 in. (2.5 mm) or less 76% of the time during the 2-month demonstration period, whereas the MSD network was accurate to within 0.1 in. (2.5 mm) 73% of the time and the unadjusted radar amounts 48% of the time. During the period when the man-machine combination had some, but not all, of the real-time rainfall data from the MSD network, 82% of all 30-min rainfall amounts of 0.1 in. (2.5 mm) or more by the man-machine combination were accurate to within 0.1 in. (2.5 mm). Thus, definite improvements in the accuracy of radar monitored rain amounts occur when limited telemetered raingage data are available to adjust the radar-indicated rain. Such improvements are most pronounced in heavier rainfall.

Evaluation of the forecasting success with the man-machine combination provided strong support for the applicability of the prediction-monitoring system to real-time operational problems in urban areas. The man-machine combination was proven superior to radar alone in providing short-term forecasts of 30 min–120 min, some 18%–32% better for individual and accumulative forecasts of 0.2 in. (5.0 mm) or greater during the 2-month demonstration period. With the addition of some telemetered raingage data in August, the man-machine combination improved substantially in accuracy. The median error of the individual 30-min, 60-min, and 120-min forecasts dropped 6%–63% between June–July and August, and for accumulative forecasts of 0.2 in. (5.0 mm) or more the median error was 63%–74% less in August than in June–July. This strongly supports our original project concept that this method can provide very useful input to the real-time operation of urban hydrologic systems.

Both monitoring and prediction information proved useful to MSD during the demonstration period. Accuracy levels were achieved that are sufficient to facilitate and improve the operational efficiency of urban systems. Further improvement in both monitoring and prediction accuracy can be expected when the system operates at full effectiveness; that is, with telemetered raingage input to the radar system for adjusting the radar-indicated rainfall field.

Recommendations

Further operation of the prediction-monitoring system with user operations

is most desirable. The 1979 demonstration period was too short to evaluate completely the system under all types of storm situations, and the data sample was not large enough to define the accuracy of the monitoring and prediction with a high degree of statistical reliability. However, the greatest need is for more operations with the system at full effectiveness; that is, with telemetered raingage data routinely transmitted and integrated into the computer computations of the radar-indicated rainfall field.

Satellite cloud data (not available in the Chicago tests) would be a useful addition to the prediction system to serve as a guide in alerting operational personnel to the expected characteristics of incoming, large-scale storm systems. Such information could help in estimating size, intensity, and movement of such storm systems many hours in advance of their arrival in the urban area.

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Analytical Cost Model for Urban Water Supply

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Introduction

Water supply systems furnish a service that is often without competition. The local water utility is ordinarily a pure monopoly except to the extent that industrial customers of water may furnish their own supplies.

Physically, it is possible to separate the water supply system into two components: (1) The treatment plant; and (2) the delivery (transmission and distribution) system (2). Each of these components has a different cost function. The unit costs associated with treatment facilities are usually assumed to decrease as the quantity of service provided increases. However, the delivery system is more directly affected by the characteristics of the area being served. The cost trade-offs between the two components will determine the least cost service area. As pointed out by Danjani and Gemmell, "Economies of scale may be offset by diseconomies of dispersion agglomeration, or spatial arrangement and pattern (6)."

Passage of the Safe Drinking Water Act has intensified interest in examining the ways in which drinking water is handled before it is delivered to the consumer (14). A primary feature of the Act is that the economics of water supply and water delivery must be considered before regulations are promulgated. A comprehensive analysis of the Interim Primary Drinking Water Regulations indicates that their economic effects on large water systems will be minor, but that there may be a potentially serious economic impact on small systems. High unit costs are generally associated with small systems and if a small system (1 mil gal/day or less is forced to install expensive treatment technology, it is conceivable that per capita costs could more than double (1).

A frequently suggested option to minimize the cost of water supply is to develop regional water utilities consisting of a group of small systems or one or more small systems combined with a larger system. The presumed economies of scale to be gained resulting from the regional system would therefore benefit all users.

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The purpose of this paper is to examine some of the tradeoffs that may exist between the economies of scale of producing water and the diseconomies of transporting it to a point of use. This analysis will be useful to planners, designers, and managers of water supply utilities.

Water Supply Economics

A characteristic of many production/transportation problems is the tradeoff between the cost of building and operating facilities to meet demands for a product and the cost of transportation (10). High transportation costs and low facility costs imply decentralization; the reverse implies a few large central facilities. These same characteristics must be considered in planning, designing, constructing, and operating water supply systems.

Water supply systems are generally composed of: (1) Acquisition facilities; (2) treatment facilities, where needed; and (3) delivery systems (8). Delivery can be further subdivided into transmission and distribution. Acquisition involves either tapping a source of water that is adequate in quantity to satisfy present and reasonable future demands on a continuous basis, or to convert an intermittent source into a continuous supply by storing surplus water for use during periods of low flows. If the water is not of satisfactory quality at the point of acquisition, treatment plants purify it, e.g., polluted water is disinfected, esthetically unattractive or unpalatable water is treated to make it attractive and palatable, water containing iron or manganese is subjected to deferrization or demanganization, corrosive water is stabilized chemically, and excessively hard water is softened. The delivery system, which constitutes the water transportation system and includes transmission and distribution components, conveys water to the consumers. Constructing and operating an urban water works system involves large investments in both operating and capital funds. The economics of the treatment/acquisition component and the transportation/distribution component are considered in the following sections.

Treatment Cost

The two main elements that comprise treatment cost are construction or capital costs and operating and maintenance cost. A number of investigators have examined treatment costs in an attempt to develop relationships between cost and the design capacity or throughput of the plant. Typical of these relationships is the following (8,12):

$$C_p = DQ_n^a \quad \ldots\ldots\ldots\

which suggests the existence of economies of scale. Similar relationships have been developed for operating and maintenance costs.

Water Transportation Cost

Water is transported and delivered to consumers through transmission pipelines and distribution mains. Transmission pipelines are the major trunk lines used to transport large volumes of water. They connect the treatment plant to the pumping station and, if gravity tunnels are not used, to the distribution system. Pumping stations, pipelines, and energy comprise the major costs of transmission. The distribution works include the meters, pipelines, and storage facilities (water tanks) necessary to convey the water from the transmission system to the customer. A pumping station might also be involved if ground-storage exists. Therefore, distribution systems include the cost of pipelines, water towers, pumping stations, and energy.

Several investigators have attempted to analyze the transportation costs associated with water supply. Coase, in his article on uniform pricing systems, examines the shape of the supply and cost curves when the same price is charged for a product or service over a given area, even though the supply cost may vary from one part of the area to another (5). He argues that the marginal costs of supply to each consumer rise with an increase in distance from the plant and that the marginal costs of supply to each individual consumer are constant at a given distance from the production source. As a result, the shape of the cost curve depends upon the quantity consumed by individuals at various distances from the plant.

Ford and Warford estimated water supply costs using area (in square miles) and Q (daily water supply) divided by area (7). They found economies of scale when Q/area increases, but diseconomies when area alone increases.

Linaweaver and Clark provided an engineering and economic basis for examining transmission costs of water (9). Two components were incorporated: capital cost of pipe and energy cost for pumping water. The energy cost depended upon the flow and distance pumped, while capital cost was determined by pipe length alone.

Total Cost Model

Logic developed in the previous section indicates that if a total cost model is developed it should consist of two component parts: Acquisition-Treatment; and Transmission-Distribution. Therefore, a total cost function could be formulated as follows:

$$C_{TOT} = AQ^{\alpha} + BQ^{\beta} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which C_{TOT} = annual cost for water supply, in millions of dollars per year; Q = Revenue-producing water pumped by a utility, in millions of gallons per year (a percentage of total water pumped); α, β = exponents; and A, B = constants.

Eq. 3 is formulated somewhat differently than those previously examined. It combines the total annual cost of each component to get total annual cost for the system, but maintains the separability of the two relationships. The

first represents the cost of treatment and the second term the distribution cost. Revenue producing water is used in Eq. 3 because it represents the most stable

TABLE 1.—Density Functions for 18 Standard Metropolitan Statistical Areas in United States (People Per Square Miles)*

Metropolitan area (1)	Variables (2)	Population 1948 (3)	1954 (4)	1958 (5)	1963 (6)
Albuquerque	λ	0.56	0.61	0.61	0.62
	K	5,748	11,387	14,148	18,180
Baltimore	λ	0.48	0.40	0.36	0.33
	K	51,159	42,693	37,481	34,541
Boston	λ	0.27	0.25	0.23	0.21
	K	35,473	32,629	28,630	24,922
Canton	λ	0.69	0.62	0.58	0.54
	K	19,994	18,724	17,610	16,591
Columbus	λ	0.78	0.65	0.58	0.52
	K	44,303	38,680	34,680	31,710
Denver	λ	0.59	0.45	0.38	0.33
	K	27,779	22,884	19,678	18,008
Houston	λ	0.37	0.28	0.24	0.21
	K	15,118	13,118	11,881	11,243
Milwaukee	λ	0.47	0.37	0.32	0.27
	K	58,318	44,262	37,823	31,123
Philadelphia	λ	0.31	0.37	0.25	0.23
	K	53,264	45,714	41,868	38,268
Phoenix	λ	0.51	0.39	0.33	0.28
	K	11,324	11,244	10,350	9,521
Pittsburgh	λ	0.27	0.25	0.24	0.22
	K	25,072	22,780	21,699	18,974
Rochester	λ	0.73	0.55	0.47	0.40
	K	39,682	28,194	24,033	20,527
Sacramento	λ	0.77	0.56	0.48	0.41
	K	22,120	18,337	16,782	15,262
San Antonio	λ	0.63	0.56	0.50	0.45
	K	27,513	28,705	25,855	23,951
San Diego	λ	0.27	0.23	0.21	0.20
	K	10,438	12,583	13,164	14,972
Toledo	λ	0.83	0.72	0.67	0.61
	K	41,123	34,661	31,768	28,151
Tulsa	λ	0.89	0.63	0.50	0.40
	K	28,788	20,126	15,339	11,947
Wichita	λ	0.98	0.74	0.63	0.54
	K	29,149	23,589	20,153	17,613

*To convert from people per square mile to people per square kilometer divide by 2.59.

measure of water production and is the cost base upon which a utility recovers its revenues.

Quantity, Distance, and Population Density Relationships

Water usage has been related to numerous independent variables, such as price, income, population, lot size, and rainfall. Water usage may be calculated for a given area at a given point in the service areas by multiplying population density be area of service and water use per capita.

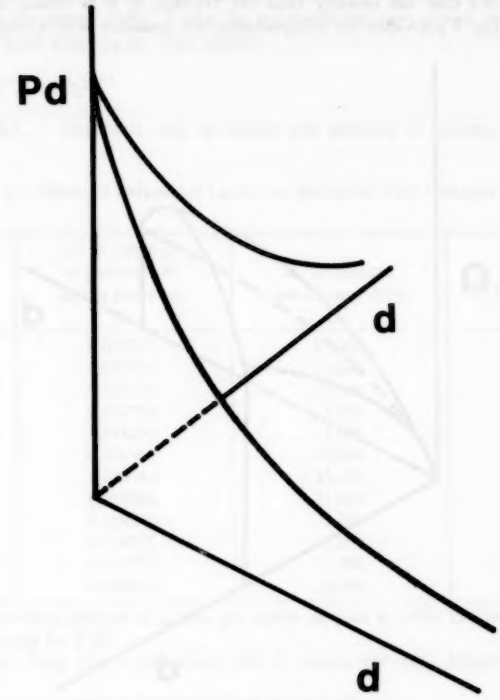

FIG. 1.—Distance Density Relationship

This relationship may be represented by

$$Q = CP_d A_r \quad \ldots \ldots \ldots

be written:

$$P_d = Ke^{-\lambda r} \quad \ldots \ldots \ldots

$$Q = \frac{2\pi CK}{\lambda^2}[1 - e^{-\lambda d}(1 + \lambda d)] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which d = limit of integration as the distance from the central reference point to the extreme point of the service area. In Eq. 7, if $d = 0$; then $Q = 0$; and if $d \to \infty$; then $Q = 2\pi CK/\lambda^2$. Q increases with d as shown in Fig. 2, and Eq. 7 demonstrates the link between Q, C, K, λ, and d.

Unit Cost Equations.—Eq. 3 can be reformulated into a unit cost equation by dividing both sides by Q. This yields:

$$\text{TUC} = AQ^{\alpha-1} + BQ^{\beta-1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

in which TUC = total unit cost, in dollars per millions of gallons. A declining

TABLE 2.—Effect of Parameter Levels on Minimum Cost Distance (Eq. 19)

λ (1)	C,[a] in millions of gallons per capita per year (2)	K,[b] in people per square mile (3)	d,[c] in miles (4)
0.4	0.054750	15,000	3.3
0.3	0.054750	15,000	4.0
0.2	0.054750	15,000	5.3
0.1	0.054750	15,000	8.7
0.2	0.018250	15,000	5.5
0.2	0.036500	15,000	5.4
0.2	0.054750	15,000	5.3
0.2	0.073000	15,000	5.3
0.2	0.054750	5,000	5.5
0.2	0.054750	10,000	5.4
0.2	0.054750	15,000	5.3
0.2	0.054750	20,000	5.3

[a]To convert from millions of gallons per capita per year to cubic kilometers per capita per year multiply by 3.785.

[b]To convert from people per square mile to people per cubic kilometers divide by 2.59.

[c]To convert from miles to kilometers multiply by 1.609.

value for TUC with distance implies continuing economies of scale over space. But, when TUC increases, economies of scale have diminished. To find the minimum cost point, one can take the partial derivative of Eq. 8 with respect to d, as shown in the following:

$$\frac{\partial \text{TUC}}{\partial d} = A(\alpha - 1)Q^{\alpha-2}\frac{\partial Q}{\partial d} + B(\beta - 1)Q^{\beta-2}\frac{\partial Q}{\partial d} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Substituting Eq. 7 into Eq. 9, taking the necessary partial derivatives and setting the result equal to 0 yields:

$$\frac{\partial TUC}{\partial d} = A(\alpha - 1)\left[\frac{2\pi CK}{\lambda^2}[1 - e^{-\lambda d}(1 + \lambda d)]\right]^{\alpha-2}[2\pi CKde^{-\lambda d}]$$

$$+ B(\beta - 1)\left[\frac{2\pi CK}{\lambda^2}[1 - e^{-\lambda d}(1 + \lambda d)]\right]^{\beta-2}[2\pi CKde^{-\lambda d}] = 0 \ldots\ldots (10)$$

It is easy to see from Eq. 10 that if α and β are less than one, this derivative will always be negative signifying a constantly decreasing TUC curve as $Q(d)$ rises. This result depends upon the relevant statistical estimates for A, B, α, and β.

TABLE 3.—Sample Calculations for Distance Cost Relationships

Q,[a] in millions of gallons per year (1)	d,[b] in miles (2)	Treatment[c] unit cost, in dollars per millions of gallons (3)	Transmission unit cost, in dollars per millions of gallons (4)	Total unit cost, in dollars per millions of gallons (5)	Marginal cost, in dollars per millions of gallons (6)
8,808.47	2	63.50	125.45	188.95	188.95
30,172.25	4	47.84	128.91	176.75	171.72
58,332.44	6	41.11	132.31	173.42	169.80
89,416.04	8	37.26	135.64	172.90	171.94
120,893.44	10	34.77	138.89	173.66	175.80
151,181.43	12	33.02	142.05	175.07	180.71
179,354.48	14	31.75	145.10	176.85	186.30
204,936.20	16	30.79	148.03	178.82	192.64
227,749.76	18	30.05	150.83	180.88	199.42
247,811.79	20	29.48	153.50	182.98	206.68

Note: Assumptions: $C = 0.054750$ millions of gallons per capita per year (0.20723 cubic kilometers per capita per year); $\lambda = 0.12$; and $K = 15{,}000$ people per square mile (5,791.5 people per square kilometer).

[a]To convert from millions of gallons per year to cubic kilometers per year multiply by 3.785.

[b]To convert from miles to kilometers multiply by 1.609.

[c]To convert from dollars per millions of gallons to dollars per cubic kilometers divide by 3.785.

Estimating Parameters for Total Unit Cost Equations.—To use Eq. 10, it is necessary to estimate the parameters in Eq. 3. Data utilized to estimate these parameters have been taken from two studies being conducted by the United States Environmental Protection Agency (3). The data used to estimate the treatment-acquisition component resulted from a 2-yr study of 12 selected utilities. Data were collected for 10 yr from at least one class A utility (revenues greater than \$500,000/yr) in each of EPA's 10 regions. The finished water from all of the utilities selected met the 1962 Public Health Service Drinking Water Standards. All of the utilities in this sample used conventional treatment and acquired water from a surface source. The estimated equation for the acquisition-treatment function is (3,13):

$$C_T = 513\,Q^{0.77}\,(R^2 = 0.61)(0.065) \quad \ldots \quad (11)$$

in which C_T = total cost of acquisition and treatment, in dollars per year; and Q = revenue-producing water in million gallons per year. Estimation of the transmission-distribution function is rather complex in relation to Eq. 3. Using data from the pipe network of one water utility, estimation of the transmission-distribution cost equation was made. This yields:

$$C_{DP} = 408\,Q_P^{0.82}\,(R^2 = 0.84)(0.026) \quad \ldots \quad (12)$$

in which C_{DP} = total cost of transmission-distribution for Q at a point in the

TABLE 4.—Sample Calculations for Distance Cost Relationships

Q_r,[a] in millions of gallons per year (1)	d_r,[b] in miles (2)	Treatment[c] unit cost, in dollars per millions of gallons (3)	Transmission unit cost, in dollars per millions of gallons (4)	Total unit cost, in dollars per millions of gallons (5)	Marginal cost, in dollars per millions of gallons (6)
2,936.16	2	81.76	152.88	234.64	234.64
10,057.44	4	61.59	157.10	218.69	212.11
19,444.19	6	52.93	161.24	214.17	209.33
29,805.42	8	47.98	165.30	213.28	211.61
40,297.91	10	44.76	169.26	214.02	216.14
50,393.93	12	42.52	173.11	215.63	222.62
59,782.85	14	40.88	176.83	217.71	228.89
68,312.23	16	39.64	180.39	220.03	236.37
75,916.77	18	38.69	183.81	222.50	244.64
82,604.12	20	37.95	187.06	225.01	253.45

Note: Assumptions: C = 0.018250 millions of gallons per capita per year (0.06908 cubic kilometers per capita per year); λ = 0.12; and K = 15,000 people per square mile (5,791.5 people per square kilometers).

[a]To convert from millions of gallons per year to cubic kilometer per year multiply by 3.785.

[b]To convert from miles to kilometers multiply by 1.609.

[c]To convert from dollars per millions of gallons to dollars per cubic kilometers divide by 3.785.

water utility service area; and Q_P = revenue-producing water at a point in the service area. In general

$$C_{DP} = BQ_P^{\beta} \quad \ldots \quad (13)$$

Eq. 13 does not correspond exactly to the transmission-distribution component of Eq. 3. Eq. 13 represents the total cost of supplying quantity, Q_P, to a point of consumption in the service area. Therefore, Eq. 13 must be integrated over the distance and area served by the utility. From Eq. 4, the total water use for a point is:

$$Q_P = CP_d\,da = (CKe^{-\lambda r})\,da \quad \ldots \quad (14)$$

in which Q_P = water use at a point in the service area, in millions of gallons per year; r = distance from the treatment plant in miles; and da = differential area.

The equation for the cost of transmission-distribution over the entire service area is derived by integrating Eq. 13 over the relevant area. This equation is as follows:

$$C_D = \int_0^{\bar{\theta}} \int_0^{d} C_{DP} r dr d\theta \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (15)$$

in which C_D = total cost of transmission-distribution for Q over the entire water utility service area; and $\bar{\theta}$ = limit of integration for service area geometry.

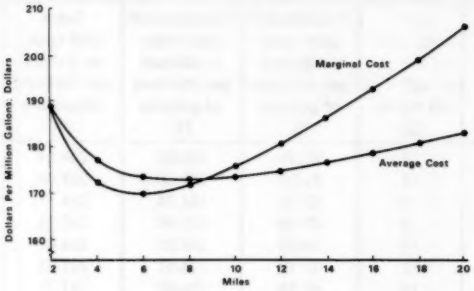

FIG. 3.—Unit and Marginal Cost with Distance using 150 gal/cap/day

Substituting Eq. 14 into Eq. 13 and performing the integration step in Eq. 15 yields

$$C_D = B \int_0^{\bar{\theta}} \int_0^{d} (CKe^{-\lambda r})^{\beta} r dr d\theta \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (16)$$

or equivalently

$$C_D = \frac{\bar{\theta} B (CK)^{\beta}}{\beta^2 \lambda^2} [1 - e^{-\beta\lambda d}(1 + \beta\lambda d)] \quad \ldots \ldots \ldots \ldots \ldots \ldots \quad (17)$$

Substituting Eq. 17 into the second term on the right hand side of Eq. 3 and Eq. 7 into the first term yields:

$$C_{TOT} = \frac{A\bar{\theta} CK}{\lambda^2} [1 - e^{-\lambda d}(1 + \lambda d)]^{\alpha}$$

$$+ \frac{\bar{\theta} B (CK)^{\beta}}{\beta^2 \lambda^2} [1 - e^{-\beta\lambda d}(1 + \beta\lambda d)] \quad \ldots \ldots \ldots \ldots \ldots \ldots \quad (18)$$

To find the total unit cost function (TUC), divide Eq. 18 by Eq. 7 to get

$$\mathrm{TUC} = A\left[\frac{C\bar{\theta}\,K\,[1-e^{-\lambda d}(1+\lambda d)]^{\alpha-1}}{\lambda^2}\right] + \frac{\dfrac{B(CK)^{\beta-1}}{\beta^2}[1-e^{-\beta\lambda d}(1+\beta\lambda d)]}{[1-e^{-\lambda d}(1+\lambda d)]} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

Taking the partial derivative of Eq. 19 with respect to d yields:

$$\frac{\partial \mathrm{TUC}}{\partial d} = A(\alpha-1)\left[\frac{\bar{\theta}\,CK}{\lambda^2}[1-e^{-\lambda d}(1+\lambda d)]\right]^{\alpha-2}[\bar{\theta}\,dCKe^{-\lambda d}] - \frac{B(CK)^{\beta-1}}{\beta^2}[1-e^{-\beta\lambda d}(1+\beta\lambda d)][1-e^{-\lambda d}(1+\lambda d)]^{-2}[\lambda^2 de^{-\lambda d}] + \frac{B(CK)^{\beta-1}}{\beta^2}[1-e^{-\lambda d}(1+\lambda d)]^{-1}[\beta^2\lambda^2 de^{-\beta\lambda d}] \quad \ldots\ldots\ldots\ldots (20)$$

Eq. 20 is the marginal cost equation with respect to distance. Using the estimated

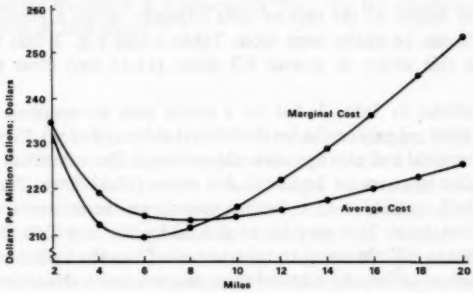

FIG. 4.—Unit and Marginal Cost with Distance using 50 gal/cap/day

values for A, B, α, and β from Eqs. 11 and 12 and appropriate values for λ, C, and K, Eq. 20 can be solved for the minimum cost distance by setting it equal to 0.

Analysis and Results

By substituting values for the parameters into Eqs. 19 and 20, the effects of distance on unit costs can be studied. Table 2 demonstrates the effects of changing the parameter values in Eq. 20. For example, lines 1–4 illustrate the effect of decreasing λ, holding C and K constant. As can be seen, d (the minimum cost distance $\partial\,\mathrm{TUC}/\partial d = 0$) increases dramatically with decreasing λ. Larger values of λ are associated with systems serving areas with population densities that fall rapidly as one approaches the suburbs. Most metropolitan areas are experiencing a transition from high but rapidly declining densities to lower but

more uniform population densities. Table 2 illustrates the effect of this phenomenon on the cost of water supply.

Lines 5–12 clearly illustrate that holding λ constant and changing C and K has little effect on the minimum cost d. Data in Table 2 show that the most important variable determining "optimal" or least-cost system size as measured by the minimum cost distance is population distribution rather than the absolute values of population density or per capita consumption.

Using the coefficients estimated for Eqs. 11 and 12, and selecting parameters for C, K, and λ to characterize an average water utility, sample calculations were conducted. The values for C, K, and λ selected for this utility were: $C = 0.054750$ mil gal/capita/yr (0.20723 km^3/capita/yr); $K = 15{,}000$ people/sq mile (5,791.5 people/km^2); and $\lambda = 0.12$. Using these, total unit costs were calculated from Eq. 19, for various distances from the treatment plant. Eq. 7 was used to estimate the change in Q with d. These calculations are summarized in Table 3. Eq. 20 was used to estimate marginal costs.

Fig. 3 shows the unit and marginal cost relationships with distance. The unit or average cost curve represents the unit cost for supplying the whole output to a service area of a given size. As distance from the treatment plant rises, or the service area expands, the incremental costs can be determined from the marginal cost curve. The marginal costs reflect the change in total costs from increased output as the service area expands, given a population density distribution. It can be easily seen from Table 3 and Fig. 3 that the minimum cost point for this utility is around 8.3 miles (13.35 km) from the treatment plant.

Table 4 is similar to Table 3, but for a utility with an assumed output based upon $C = 0.018250$ mil gal/capita/yr (0.069076 km^3/capita/yr). Figure 4 presents the relevant marginal and average cost relationships. The minimum cost distance for this situation appears to be about 8.5 miles (13.68 km). The relationship shown in Table 2, namely, that as output goes down the minimum cost distance rises, is born out here. This may be explained by the fact that the utility with the lower output is still obtaining returns to scale from the treatment component which continues to offset the transmission diseconomies. However, this effect only adds 1–0.2 of a mile (1.61 km–0.32 km) to the minimum cost distance. It is obvious that the smaller utility has a significantly higher unit cost over the entire range of distances.

Summary and Conclusions

Equations have been developed that show the relationship between the minimum cost point for a water distribution system and distance from the point of production. The analysis is based on an assumption relating population distribution to distance from a central point within a city. By developing a deterministic relationship between total cost and the cost of acquisition-treatment and transmission-distribution in terms of water produced and transport distance, it is possible to find a point at which system economies of scale are lost. This is defined as the point at which the partial of the average cost function with respect to d is zero. Before this point, the average cost with respect to distance is decreasing (increasing economies of scale) and after this point is increasing (decreasing economies of scale). Obviously values for α and β will have a

significant effect on the minimum cost point.

Setting a partial of the average cost equation equal to zero yields a system size for minimum cost. The effect of population distribution, per capita consumption, and absolute levels of population density have been studied. It has been concluded that per capita consumption and absolute levels of population density have little effect on least cost system size. However, the minimum cost solution is very sensitive to population distribution. Therefore, changes in population dispersion have a critical effect on the point at which economies of scale are lost in a water system. It is hoped that water supply planners, designers, and managers will consider these data with care when making decisions concerning regionalization of water supply. Although certain advantages exist to the regional concept, as illustrated by this analysis, it has limitations and disadvantages as well. The purpose of this paper is to emphasize the tradeoffs that must be considered in developing regional strategies. Each situation must be evaluated separately with respect to the benefits and costs of regionalization.

Acknowledgments

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Appendix I

Using Eqs. 19 and 20, costs for a selected midwestern water utility were examined to verify the equations' applicability. Table A-5 shows the parameter values assumed for the utility, the actual budget values, and also contains the estimated total treatment acquisition and transmission-distribution cost for the utility. The flexibility of this model is evident here in that it will handle most service area configuration. The equations were based upon a circular service area, but not all utility service areas possess such a shape. However, the equation can easily be adjusted to conform to noncircular service areas, such as a semicircle or pie slice. This can be done by adjusting the 2π limit of integration to a more appropriate value. In fact, for the selected water utility, the limit of integration actually corresponds closer to 0.4π instead of 2π. Using this new limit of integration, Q and cost can be more accurately estimated. As can be seen, Eqs. 19 and 20 yield fairly close estimates of the total annual costs for the critical components (4).

Because Eq. 5 upon which Eqs. 10 and 20 is derived to be a total cost function versus Q, Eqs. 19 and 20 can be used to estimate various scenarios of demand for water supply. To illustrate the use of Eqs. 19 and 20, a growth scenario (see Table A-6) was assumed for 10 yr for a hypothetical utility. The growth scenario over the 10-yr period is illustrated by declining values of λ (column 2), increasing per capita consumption (column 4), expansion of the service area (column 3), and increasing total Q (column 5). Columns 6 and 7 are the hypothetical component costs. As can be seen from column 10, the total unit cost is declining as has been seen in many actual utility studies. This analysis provides a further verification of the model's usefulness and validity.

TABLE A-5.—Budgetary Calculations

C_t,[a] in millions of gallons per capita year (1)	K_t,[b] people per square mile (2)	λ (3)	Q_t,[c] in millions of gallons per year (4)	Estimated treatment-acquisition cost, in dollars per year (5)	Estimated treatment-distribution cost, in dollars per year (6)	Actual treatment-acquisition cost, in dollars per year (7)	Actual treatment-distribution cost, in dollars per year (8)
0.054750	15,000	0.12	49,6000	2,116,597	7,605,031	2,009,977	6,446,962

[a]To convert from in millions of gallons per capita per year to cubic kilometers per capita per year multiply by 3.785.
[b]To convert from people per square mile to people per cubic kilometer divide by 2.59.
[c]To convert from millions of gallons per year to cubic kilometers per year multiply by 3.785.

TABLE A-6.—Growth Scenario

Year (1)	λ (2)	d_t,[a] in miles (3)	C_t,[b] in millions of gallons per capita per year (4)	Q_t,[c] in millions of gallons per year (5)	Total A-T cost, in dollars per year (6)	Total T-D cost, in dollars per year (7)	Unit A-T cost, in dollars per millions of gallons (8)	Unit T-D cost, in dollars per millions of gallons (9)	TUC, in dollars per millions of gallons (10)
1	0.3	7.0	0.018250	11,856	703,153	2,169,356	59.31	182.98	242.29
2	0.3	7.2	0.027385	18,220	978,903	3,113,303	53.73	170.87	224.66
3	0.3	7.4	0.036500	24,855	1,243,329	4,050,959	50.02	162.98	213.00
4	0.2	7.6	0.040150	42,461	1,877,889	6,424,109	44.23	151.29	195.52
5	0.2	7.8	0.043800	47,684	2,053,356	7,127,497	43.06	149.47	192.53
6	0.2	8.6	0.047450	53,113	2,231,115	7,853,036	42.01	147.86	189.87
7	0.1	8.2	0.051100	95,557	3,506,869	12,936,492	36.70	135.38	172.08
8	0.1	8.4	0.054750	106,118	3,801,674	14,218,365	35.82	133.99	169.81
9	0.1	8.6	0.058400	117,192	4,103,636	15,552,887	35.02	132.71	167.70
10	0.1	8.8	0.062050	128,779	4,412,635	16,940,088	34.26	131.54	165.80

[a]To convert from miles to kilometers multiply by 1.609.
[b]To convert from millions of gallons per capita per year to cubic kilometers per capita per year multiply by 3.785.
[c]To convert from millions of gallons per year to cubic kilometers per year multiply by 3.785.

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Appendix III.—Notation

The following symbols are used in this paper:

A = constant in total cost equation;

a = elasticity of total cost with respect to design capacity (with economies of scale $a < 1$);

A_r = area over which water usages is to be determined;

B = constant in total cost equation;

C = per capita water use, in millions of gallons per year;

C_D = total cost of transmission-distribution for water over entire water utility service area;

C_{DP} = total cost of transmission-distribution for water at point in utility service area;

C_P = total capital cost of complete water treatment plant, in dollars;

C_T = total cost of acquisition and treatment, in dollars per year;
C_u = unit capital cost, in dollars per million gallons;
D = constant in cost equation;
d = limit of integration as distance from central reference point to extreme point of service area;
d_a = differential area;
e = base of natural logarithm;
K = population density in people per square mile at an initial point;
P_d = population density at r distance from initial point;
Q = revenue producing water pumped by utility, in millions of gallons per year for a given area;
Q_n = design capacity of a plant, in millions of gallons per day (MGD);
Q_p = water use at point in service area, in millions of gallons per year;
r = distance from central point in miles;
TUC = total unit cost, in dollars per millions of gallons;
β = exponent in total cost equation;
λ = percentage change per mile;
∂ = exponent in total cost equation; and
θ = limit of integration for service area geometry.

Salinity Forum: What? How? Why?[a]

By Daniel F. Lawrence[1] and Barry C. Saunders,[2] Members ASCE

Introduction

One of C. Northcoate Parkinson's most perceptive laws is that "work in an organization will expand to match the time available." For this reason, it is sometimes difficult to distinguish between *workable* water management institutions and water management institutions which merely exist. Institutions, like problems, are easier to create than to terminate.

In order for a management institution to be *workable*, two conditions must be met. First, there must be a real problem to address. Without a continuing reason for existence, even the best conceived, brilliantly structured, and perfectly staffed organization is doomed to drift aimlessly.

Second, and equally important, is the requisite that parties in the institution must be committed to work towards a solution of the problem. Voluntary interstate organizations necessarily function through consensus. Since compromise is the essence of consensus, all parties must be willing to compromise to some degree in order to achieve the desired end.

The Colorado River Basin Salinity Control Forum qualifies in both aspects—purpose and commitment. The following sections will describe the salinity problem on the Colorado River, the history of interstate cooperation (and conflict), how the Forum came about, and what has been accomplished to date.

Description of the Salinity Problem.—Salinity is not unique to the Colorado River. Although all precipitation is substantially free of dissolved solids, from the moment it touches the ground, water begins to dissolve and transport salts. Salinity can be measured in salt load (e.g., tons/day) or concentration (e.g., milligrams/liter). The salt load can be increased (decreased) only by adding (removing) salt from the system. The concentration, on the other hand, can be increased (decreased) by adding (removing) salts to the system, or by removing (adding) dilution water. (This difference is critical, as will be seen in later sections.)

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Salinity can also be categorized as to cause: natural or man-induced, and source: point or diffuse. Both these attributes have important impacts on regulatory and technical aspects of salinity control. In the Colorado River Basin, estimated origin of salinity is (in terms of salt concentration at Hoover Dam):

Natural diffuse sources:	39%
Natural point sources:	8%
Irrigated agriculture	37%
Municipal & industrial	1%
Exports out of the basin	3%
Evaporation & phreatophytes	12%
Total:	100%

The Colorado River Basin lies in the southwestern part of the United States and covers substantial portions of Arizona, California, Colorado, Nevada, New Mexico, Utah, and Wyoming (Fig. 1). The basin is about 244,000 square miles, (632,000 km^2) and ranges in elevation from about 100 feet (30 m) at the Mexican border to over 14,000 feet (4,300 m), in the Rocky Mountains of Colorado.

There is relatively little inflow to the Colorado River below Lee Ferry (near the Arizona-Utah border); yield in the lower basin is only about 17% of the virgin flow at Lee Ferry. At this point, average annual runoff for virgin conditions is between 14,000,000 and 15,000,000 acre-feet (17,200,000,000–18,500,000,000 m^3). The vast majority of this flow comes from melting snow, and without storage, would occur in the months of April–July. The river is essentially regulated, with mainstream storage reservoirs behind Hoover and Glen Canyon dams, and major reservoirs on the Green, Gunnison, and San Juan rivers. Total usable capacity is over 61,000,000 acre-feet, (75,000,000,000 m^3).

By far the largest use of Colorado River Basin water is for irrigated agriculture (about 7,700,000 acre-feet, 9,500,000,000 m^3). Municipal and industrial use accounts for about 1,700,000 acre-feet, (2,100,000,000 m^3). Although there is little additional water to be developed in the lower basin, the upper basin states, Utah, Wyoming, Colorado, and New Mexico, are expected to increase their use of Colorado River water by at least 1,500,000 acre-feet, (1,850,000,000 m^3) by the year 2000.

A major economic cost of salinity is that associated with irrigated agriculture, but research conducted to develop salinity level versus damage curves has not been completely successful. There is disagreement as to whether there is a threshold level, but all accept the fact that salinity levels in excess of 600 mg/L–700 mg/L require special management practices to avoid damage to most crops. Damage caused by salinity in municipal water is much more difficult to quantify, but it is also quite significant. Most assuredly, high salinity levels reduce service life and increase maintenance costs of water-using appliances and pipe systems.

A seemingly unrelated factor—the Mexican Water Treaty—has had a great impact in forcing political recognition of the salinity issue. Although the treaty made no specific mention of water quality, the Mexican government became very concerned with increasing salinity levels in the water crossing the United States-Mexican borders. Pressure was brought to bear on the White House,

and in 1973 an agreement was reached (Minute 242 of the International Boundary and Water Commission) which required salinity levels of the water delivered to Mexico to be no more than 115 ppm ± 30 ppm greater than the water delivered to lower basin users at Imperial Dam.

The 1972 Amendments to the Clean Water Act (Public Law 92-500) set the

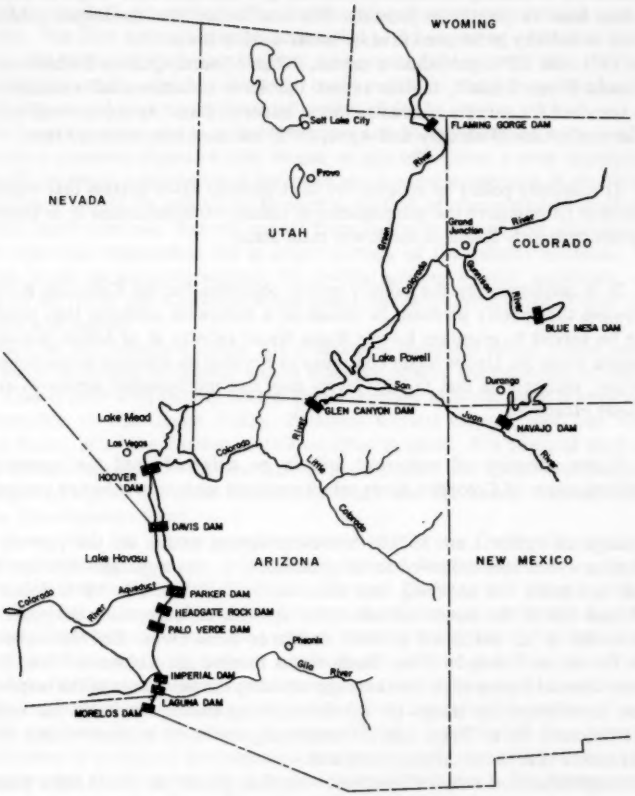

FIG. 1.—Colorado River System

final stage for real action on the salinity problem. The Environmental Protection Agency (EPA) interpreted Section 303 of this act as requiring that numeric criteria be set for salinity on the Colorado River. In 1973, EPA submitted, to several of the states, its preliminary views on this matter, including a suggestion for creation of an interstate organization to develop a salinity control plan.

Creation of Forum.—While the Mexican problem and clean water legislation had been developing, water resources and water quality leaders in the Colorado River Basin states had not been oblivious to the situation. In 1960, under provisions of the original Federal Water Pollution Control Act, the Conference in the Matter of the Pollution of the Interstate Waters of the Colorado River and its Tributaries was formed. This group identified data deficiencies as a major impediment to dealing effectively with the problem and took steps to improve the data base as quickly as possible. The conference also developed guidelines related to salinity to be used in state water quality laws.

In 1971, the EPA published a report, "The Mineral Quality Problem in the Colorado River Basin"; in this report the EPA recommended establishment of a standard for salinity concentration at Imperial Dam. At the seventh session of the conference (February and April, 1972) the members resolved that:

> 1) a salinity policy be adopted for the Colorado River system that would have as its objective the maintenance of salinity concentrations at or below levels presently found in the lower main stem;
>
> 2) in implementing the salinity policy objective for the Colorado River system the salinity problem be teated as a basinwide problem that needs to be solved to maintain Lower Basin water salinity at or below present levels while the Upper Basin continues to develop its compact-apportioned water, recognizing that salinity levels may rise until control measures are made effective;
>
> 3) the adoption of numerical criteria be deferred until the potential effectiveness of Colorado River salinity control measures is better known.

Passage of Public Law 92-500, however, forced action on the part of the conferees which they believed to be premature. It was apparent that the EPA would not settle for anything less than numerical standards by October 15, 1975, and that if the states did not come up with an acceptable proposal, the EPA would in all likelihood publish standards themselves. On November, 9, 1973, the seven Colorado River Basin states formed the Colorado River Basin Salinity Control Forum with members appointed by the governors of the respective states, to address the issues of: (1) Establishing numeric criteria for salinity in the Colorado River Basin; and (2) developing a plan of implementation which would insure that those criteria were met.

Accomplishment of Initial Objectives.—At first glance, it would have seemed that the forum was faced with an impossible task. Since the EPA seemed to be firmly committed to standards of salinity concentration, at then existing levels, not only could no salts be added to the river system, but no dilution water could be removed. The Forum could easily have been polarized, since the lower basin had an economic stake in maintaining present salinity levels, while the upper basin had a minimal salinity problem, but wanted to remain free to develop remaining unused compact allocations.

One of the first steps to be taken was the establishment of a work group which could concentrate on technical aspects of the problem, leaving the Forum

free to negotiate around the seemingly insurmountable obstacle. The Forum and work group enlisted the assistance of the Bureau of Reclamation (now Water and Power Resources Service) to determine impacts of proposed upper basin developments on salinity in the lower mainstem Colorado River.

Over time, it became apparent that maintaining present salinity levels while allowing further water development could be accomplished by a two-pronged approach: (1) Minimizing salt pickup from future developments; and (2) reducing wherever practical, present salt inflow to the system from natural and man-induced sources. The first action could be accomplished by imposing additional design criteria for proposed irrigation projects, and presented relatively little problem. The second action, which potentially could result in much larger reductions of salinity, would require a substantial outlay of funds.

The fact that seven states with 14 senators and 59 congressmen were working towards a common objective now began to pay off. From a state standpoint, federally imposed controls could be made much more acceptable if the federal government would be willing to finance a portion of the activities necessary to meet those controls. Natural runoff from federally owned and administered lands was also responsible for a major portion of the salinity problem. The Forum began to generate support for federal salinity control legislation, and on June 24, 1974, the Colorado River Basin Salinity Control Act (Public Law 93-320) was passed.

Title I of the Salinity Control Act authorized measures which would ensure that water deliveries to Mexico would meet the quality requirements of Minute 242. Title II dealt with salinity control projects above Imperial Dam; it authorized construction of the Grand Valley, Paradox, Crystal Geyser, and Las Vegas Wash Units, and investigation of twelve other projects. The costs of each unit were to be financed 75% by the federal government, and 25% from the states through the Upper Colorado River Basin Fund and the Lower Colorado River Basin Development Fund.

In June 1975, the Forum completed the draft "Water Quality Standards for Salinity Including Numeric Criteria and Plan of Implementation for Salinity Control," commonly called the Forum Report. The numeric criteria included were the 1972, flow-weighted, average annual salinity concentrations for below Hoover Dam, below Parker Dam, and at Imperial Dam. The EPA had allowed little room for manuever in these numbers, despite the fact that some Forum members felt that salt loading would have been a much more equitable parameter. With standards fixed to tonnage of salt, additional beneficial consumptive use would not have been restricted. Passage of Public Law 93-320 then permitted finalization of a plan of implementation consisting of state and federal control measures that would permit these standards to be maintained. The Forum-recommended numeric criteria and plan of implementation were adopted by the states and approved by the EPA.

Continuing Work of Forum.—The responsibilities and tasks of the Forum certainly did not end with publication of the Forum report. The EPA intended that the Forum assume a major role in insuring that the plan of implementation progressed in a timely manner. A section on responsibility for accomplishing salinity control measures was included in the Forum report, indicating Forum responsibility for coordination of matters requiring state action.

A major activity of the Forum is monitoring federal, state, and local salinity

control programs. At the present time, this includes actions of the Water and Power Resources Service, Bureau of Land Management, Department of Agriculture, and 208 local agencies. Closely related to this is keeping congress informed of legislative and fiscal requirements for salinity control.

On several occasions the Forum has seen a need to develop policy statements. The most notable of these to date has been concerning the National Pollutant Discharge Elimination System (NPDES). The Forum on February 28, 1977 adopted the "Policy for Implementation of the Colorado River Salinity Standards through the NPDES Program." In this statement, the Forum affirmed a policy of "no-salt return whenever practicable," and elaborated on specific requirements for exception.

A very critical and time consuming activity of the Forum members recently was assisting the EPA and Justice Department in their legal defense of the EPA approval of the salinity standards adopted by the states (EDF versus Costle). In this action, the Environmental Defense Fund contended that the EPA acted illegally in approving the state adopted standards. An interesting aspect associated with the case was EDF's surprise that the State of California chose to support the EPA rather than EDF in the litigation. All seven states were defendants in the case.

Although much progress in salinity control has been made, from a practical standpoint, a great deal remains to be done. Opportunities for actual removal of salt from the river system are few and far between and even the best of these, when subjected to the spotlight of detailed feasibility investigation, turn out to be extremely costly, and often develop unforeseen complications. Crystal Geyser and Las Vegas Wash would certainly be examples of this. Salt pickup from rangeland is a significant source of salinity, and as yet, we have not developed acceptable methods of control. Even when the technical solutions are developed, economics and politics must still be dealt with. It is hoped, though, that what made the forum successful in the past will continue to do so in the future.

Reasons for Forum Success as an Institution.—It cannot be too strongly stressed that a necessary, though not always sufficient, reason for success of an institution or organization is a clear mission or purpose. Certainly the passage of the Clean Water Act and the posture of the EPA provided this direction.

There was probably no positive force for cooperation in the upper basin, since salinity was not a major problem yet, and would not become one in the foreseeable future. But water leaders in the upper basin could see probable actions by the EPA leading to inhibition of water development and use and perhaps even litigation and political action by lower basin states, if agreement were not reached.

In the lower basin, there were positive forces for cooperation in addition to those present in the upper basin. Water users in the lower basin were presently experiencing damages related to salinity, and these could be expected to increase with upper basin water development.

On the surface, it appeared that the lower basin had no real reason to compromise with the upper basin, and could afford to sit back and let the EPA enforce stringent salinity standards. From a practical standpoint, however, the lower basin realized that legal or political actions to restrict upstream development would not succeed without extensive litigation, and would not result in salinity

reduction for many years. They saw that cooperation was the key to salinity control and management.

Finally, the members on the Forum were key people who could count on political support for whatever decisions they made regarding salinity. When the mission is clear, when the stakes are high, and when the participants are powerful, there will be action, and the organization will succeed.

Developing Dam-Break Flood Zone Ordinance

By George F. McMahon,[1] A. M. ASCE

Introduction

In the United States there has been interest in implementing dam-break flood ordinances, similar to existing flood ordinances. In Georgia an interagency task force has been assembled to develop a model dam-break flood hazard ordinance for counties and municipalities, and to adapt the ordinance to Gwinnett County, Georgia. The task force is known as the Georgia Safe Dams Task Force, and its specific responsibilities are to:

1. Develop guidelines for determination of dam-break flood zones.
2. Develop a model dam-break flood zone ordinance for counties and municipalities in the State of Georgia.
3. Adapt the model ordinance to Gwinnett County.
4. Inform Gwinnett County officials of dam-related problems.
5. Map all dam-break flood zones in Gwinnett County.
6. Assist other counties and municipalities in adapting dam-break flood plain ordinances.
7. Evaluate the effectiveness of the Gwinnett County ordinances.

The agencies participating on the task force are:

1. State of Georgia: (a) Office of Planning and Budget (OPB); (b) Soil and Water Conservation Committee (SWCC); and (c) Environmental Protection Division (EPD) of the Department of Natural Resources (DNR).
2. Gwinnett County: Resource Conservation and Development District Council (RC&D).
3. United States Government: (a) Soil Conservation Service (SCS) of the Department of Agriculture; and (b) Army Corps of Engineers (Corps).

The purpose of the model ordinance is to control development downstream of Category II dams, as categorized by the State of Georgia (15), such that these dams remain in a low hazard category. Category II dams are defined

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as dams where improper operation or failure would not be expected to result in loss of human life. Category I dams are dams where improper operation or failure would be expected to result in loss of human life. The ordinance differs from flood ordinances established under the Federal Flood Insurance Program in several important respects. One difference is that the Flood Insurance Program deals with both existing and future flood plain development, while the proposed ordinance was developed only to control future development and maintain low-hazard classification of dams. The ordinance does not define eligibility for flood insurance for dam-break floods, since it is intended to restrict development in essentially undeveloped (consequently low-hazard) areas below dams. Another important difference is that the ordinance is intended, primarily, to prevent loss of life, and, secondarily, to prevent damages. The Flood Insurance Program was developed for protection against property losses due to floods.

This paper presents the guidelines used to establish dam-break flood hazard zones in the State of Georgia, and discusses the level of effort required to analyze dams in Gwinnett County. Problems and time requirements of State personnel delineating dam-break flood zones using the guidelines are explored. The draft model ordinance developed by the task force is also presented.

Guidelines

Purpose.—The purpose of these guidelines is to establish criteria for reasonably accurate determinations of inundated areas due to dam failures. Efforts were made while developing the guidelines to recommend methods that are the theoretically applicable, are reasonably easy to apply to large numbers of dams, and which will withstand legal and technical scrutiny for purposes of amending dam-break flood hazard zones determined by the State. These guidelines will apply to some of the most important aspects of dam-break flood forecasting, namely:

1. Assumptions for the mode of failure, i.e., the geometrical and temporal description of the breach.
2. Computation of the outflow discharge hydrograph produced by the breach.
3. Routing of the outflow hydrograph through the downstream valley in order to determine peak water-surface elevations, discharges, and travel times (if required) (10).
4. Determination of realistic initial and boundary conditions for dam-break flood-wave propagation.
5. Assumptions to be made in cases of domino-type failures.
6. Data required by recommended methods.

These guidelines reflect an effort to provide methods, which will yield accurate results commensurate with the purposes for which they are applied. In addition, the methods recommended herein are readily available, well documented, comparatively tested, nonproprietary, and have sufficient user subscription to provide a wide variety of experience data. Procedures presented in these guidelines represent combinations of theoretical methods, which are felt to be applicable in determining peak stages due to dam-break floods with a reasonable degree of accuracy and with a minimum of computational effort. The primary information

required for dam-break flood zoning is the maximum water surface profile expected from a hypothetical dam failure. Details of the wave-front motion are of concern only insofar as they contribute to the accuracy of profile determination. Many details often must be ignored "a priori" because there are no reasonable known methods for handling them, such as effects of debris buildup, channel erosion, and bridges. Guidelines presented herein are for the purpose of reasonable estimation of water-surface profiles downstream of ruptured dams, using assumptions based on a variety of field observations, model testing, and model applications. The following sections summarize many important aspects of previous research on dam failure flood routing.

Mode of Failure.—Failure mechanics for earth dams are not presently well understood. Efforts by Cristofano (5) to describe breach formation geometrically and temporally require critical assumptions to be made. Therefore, for reasons of simplicity, wide applicability, and uncertainty in the failure mechanism, assumptions for breach formation should be based on experience data and the effects of such assumptions.

Assumptions adopted for breach geometry of earth or rock-fill dams are based on case histories for earth, concrete gravity, and arch dams (18), and on assumptions adopted by the United States Army Corps of Engineers. (7) Due to observed similarities of final failure shapes caused by overflowing or structural failure, such as piping, no distinction is provided in these guidelines. In accordance with observed data, the adopted breach shape should be approx. four times the dam height at the top, and twice the dam height at the bottom, yielding breach side slopes of $1V$ to $1H$. For concrete portions of combination dams or concrete dams, the breach should be assumed to be total.

Studies based on rate of breach formation mechanics (2) show that sediment transport analysis using even a relatively simple empirical relationship developed by Schoklitsch (26) would require more information than is easily obtainable about most (if not all) Category II dams. Sediment transport analysis is further complicated by nonhomogeneity of embankment fill material, types of materials used, and variation in compaction. In addition, any model using sediment transport equations would of necessity be continuous, i.e., would vary transport rate in time along with velocity, hydraulic gradient, and breach geometry, which are in turn affected by these factors. Therefore, assumptions concerning rate of breach formation should be formulated based on significant hydraulic effects and study objectives. It can generally be stated that for larger reservoirs (large enough so that failure can occur in less time than required for the negative wave to travel to the upstream end of the reservoir and return), the assumption of instantaneous, partial failure does not significantly affect peak stages resulting from breach outflow hydrographs. Another way of presenting this concept is to assume the failure takes place in such a manner that a relatively insignificant amount of lake storage is evacuated during the failure time, say 5% (9). The author's experience with dam failure analysis for large dams on the Savannah River shows negative wave one-way travel times on the order of 1 h (reservoir lengths of 40 mile); therefore, a failure time of 2 h or less could be considered instantaneous.

Whether or not a breach occurs instantaneously, downstream channel storage significantly reduces the impact of peak discharge on peak stages. Therefore, in most cases, instantaneous breaches (from a hydraulic standpoint) are possible

and should be specified. Since most numerical solution techniques for dynamic wave propagation require a finite time to peak of boundary hydrographs, the following procedures are provided for estimation of failure times. A minimum breach time, based purely on momentum considerations, can be formulated in the following manner, assuming discharge increases linearly in time to a maximum:

$$\frac{t \geq 2\gamma_F \forall_F}{Qp} \quad \ldots\ldots\ldots\

$$t \leq \frac{1{,}008\,SA}{\bar{B}H^{1/2}} \quad \ldots \ldots \ldots

dams) or extremely long or dendritic reservoirs where the level pool assumption of storage routing could produce significant error in computed outflows. It is the author's experience with singular failure of large dams on the Savannah River impounding several million acre-ft for a river length of 40 mile that the differences in downstream peak stages produced by storage or dynamic reservoir routing techniques are negligible. In cases where the breach is less than 50% of the total dam, the negative wave moving upstream through the reservoir is nearly negligible, allowing the level pool assumption. This was noted by Brown (6) and others analyzing photographs and films of the Teton failure. Therefore, in most cases, storage routing is an acceptable method of determining outflow hydrographs.

In order to perform a storage routing through the reservoir a breach rating curve must be developed. A rating for a trapezoidal weir, neglecting submergence effects is easily computed, but may not be valid after the reservoir is depleted to less than 1.5 times the tailwater depth. Submergence can be taken into account using Venard's (29) correction factors for the broad-crested weir equation, although this would require continuous recalculation of the correction factor based on the numerical solution of tailwater elevations. The National Weather Service Dam-Break model, as described by Fread (10), computes breach outflow while accounting for breach geometry (trapezoidal or rectangular), time-rate of breach formation, and submergence effects, and in this respect is more versatile and realistic than most other methods for developing breach-outflow hydrographs. This model can be used with dynamic or storage routing through the reservoir.

It is difficult to assess the effect of various methods of developing breach-outflow rating on downstream peak stages. Since the National Weather Service model provides for storage and dynamic reservoir routing, time rate of breach formation, and submergence effects, its usage is recommended for most dam failure studies. This recommendation is further supported in discussion of downstream routing techniques and other criteria presented in this paper. An exception to this recommendation would be a very large reservoir with significant tributary storage, subject to severe flooding from upstream dam failures. Studies conducted by the author show significant variation in wave timing and resulting peak reservoir elevations when attempting to route large dam-break floods into dendritic reservoirs using nonlinked (single-channel routing) dynamic methods as opposed to linked (multiple-channel routing) methods. It is not expected this situation will occur in analyzing Category II dams.

In general, storage routing should be used through the reservoir (excepting some domino-type failures) because of greater ease in representing reservoir volume than with dynamic routing. Storage routing can be used with only a few reservoir elevation-surface area points, whereas great care must be exercised to assure reservoir cross sections correctly reflect variation of volume and conveyance with elevation.

Routing Breach Outflow Hydrograph.—As with reservoir-flood routing, methods for routing the flood hydrograph through the downstream channel can be classified as essentially hydrologic or hydraulic. There is a great deal of literature dealing with relative accuracy, advantages, and disadvantages of many methods; however, only general conclusions relating to their effectiveness in computing peak stages are appropriate for these guidelines. There are some simplified dam-break flood methods that are not considered to be generally applicable.

A method developed by the Defense Intelligence Agency (4) and evaluated by Gundlach and Thomas (17) calculates the outflow hydrograph at the dam axis, but does not route it downstream. The Dimensionless Graphs method presented by Sakkas (25) was developed from specialized numerical solutions of the one-dimensional continuity and momentum equations applied to dry, prismatic channels using the method of characteristics. Knowing the reservoir volume, initial elevation, valley slope and roughness, and a simplified channel cross section at the dam, the user can determine time of wave-front arrival, maximum-depth profile, and time of peak depth using dimensionless curves. However, applicability of the method is severely restricted by the assumptions made in developing the curves, namely:

1. The entire dam fails instantaneously.
2. The downstream valley is prismatic and can be represented using a single, average slope and roughness.
3. The downstream valley is dry.

In addition, the analysis of large dams or long downstream valleys would extend beyond the range of the curves, and the breach outflow hydrograph developed in accordance with the previous section is not utilized. Another method, developed by the Soil Conservation Service (1) and referred to as the Att-Kin model, is not recommended due to its limited applicability to dry-bed assumptions and empirical simplification of breach outflow hydrographs.

The most commonly used hydrologic techniques for flood routing are Modified-Puls, Muskingum, Working *R* and *D*, and Straddle-Stagger. These methods often rely upon rating data compiled using hydraulic techniques such as steady-flow backwater computations, kinematic waves, or weir and orifice equations.

There have been many evaluations of Modified-Puls storage routing applied to dam-break floods, and analysis will be confined to this technique, as it is felt to be applicable to most hydrologic routing techniques. Studies by Land, (21) Gundlach and Thomas, (17) Keefer and Simons, (20) and Strelkoff, et al, (27) suggest the following limitations to Modified-Puls (MP) routing:

1. MP works best for conditions for which it was developed, namely very gradually varying flows in reservoirs or prismatic channels. It does not account for the relatively large acceleration effects of dam-break flood waves.
2. MP does not compare well with dynamic methods for more rapidly varying flows in nonprismatic channels, as are most frequently encountered in dam-break flooding. MP is particularly inaccurate in the immediate vicinity of the dam.
3. MP is often limited by the applicability of rating data developed using kinematic wave, steady state backwater, or rating equation approaches to more rapidly varied, high-flow conditions most frequently encountered in dam-break floods.
4. MP requires similar input information to that required by some dynamic methods. Generally, Computer costs for MP models are lower than for dynamic models; however, most often computer costs comprise a very small portion of the total cost in evaluating dam-break floods.

Due to these limitations and relatively insignificant cost advantages in application, MP routing is not recommended. Some MP models may prove more difficult

and time-consuming to use because of the necessity to develop rating data prior to performing the routing. There are well-documented and widely-used MP models, such as HEC1DB (8). However, this model requires as much or more computer core memory as do most dynamic routing models now available. HEC1DB is most applicable in dam safety investigations, as it performs watershed runoff determination and routing in addition to its dam-break flood routing capabilities.

Hydraulic techniques can generally be classified as kinematic, diffusion, or dynamic, depending upon what terms in the one dimensional momentum equations are included in their formulation. There are some methods, such as the multiple-linearization models developed by Keefer and McQuivey (19) which combine aspects of hydrologic and hydraulic methods to develop channel response parameters to driving flood waves. However, this model has not been widely used and does not directly include reservoir routing as well as temporal and geometric description of the breach. In addition, the model requires rating data as do MP methods, and, therefore, is not recommended.

Since solution of the kinematic and diffusion wave equations by computer requires essentially the same input data as full dynamic solutions, there are no significant advantages to utilizing such methods. Therefore, they are not recommended. In addition, to the best of the author's knowledge, there are presently no numerical models developed using kinematic or diffusion equations specifically for dam-break floods.

One-dimensional dynamic-wave equations have been formulated many different ways and solved using various numerical and graphical techniques. The most common formulation is attributed to de Saint Venant, (6) and has been solved in various forms using the method of characteristics, explicit, implicit, and combination finite-difference techniques, as well as finite-element techniques. The applicability of the St. Venant equations to dam-break waves (including bore propagation) has been demonstrated by Martin and Zovne (22). Most work to date shows that the accuracy of computed peak stages is not significantly affected by whether or not the solution incorporates a technique for simulation of bore propagation. Therefore, this is not considered to be a deciding factor in selection of an applicable routing procedure. Most dynamic routing techniques compute peak stages with accuracy commensurate with channel geometry, roughness, and boundary conditions specified as input. In order to select a model for general application, a weighted decision matrix was prepared for the following models: HEC1DB (8) USTFLO, (16) DAMBRK (13) and MOC-LIF (3). These four models have been fairly well documented and tested; some have been used extensively. While there are many other models for unsteady flow simulation, they were not felt to offer any significant advantages over these four, and are generally proprietary, not documented or readily available. The criteria for acceptance are arranged in order of decreasing importance. Each model is assigned a number from 1–10 is each category, with 10 signifying "best." Weighting factors for each criterion are established at the author's discretion, and ratings reflect comparative studies cited in these guidelines and the author's opinions. Model descriptions are shown in Table 1, and the selection matrix in Table 2.

As shown in Table 2, the National Weather Service DAMBRK model is generally the best to apply for routing dam-break floods; therefore, it is recommended.

Because the model can use irregularly spaced channel cross sections and reservoir storage routing, many dam-break flood profiles can be computed using only three or four valley sections and minimal reservoir elevation storage data. The model, due to its implicit solution scheme, is very efficient in computation time. It will route domino-type failures in a single run, and will handle variable breach geometry and time rate of breach formation for each dam. If desired, the model can account for downstream bridges and culverts by including these structures as multiple dams. This model is the most robust (numerically stable) model of those evaluated with the exception of HECIDB. It will internally add interpolated cross sections to maintain dynamic stability. Static and dynamic

TABLE 1.—Model Description

Model (1)	Solution technique (2)	Remarks (3)
HEC1DB (Corps of Engineers)	Modified-Puls-kinematic wave	Well documented, easily applied, incorporates variation in breach geometry and time-rate of breach formation. Presently limited to single dam applications with dry downstream channels.
USTFLO (Corps of Engineers)	Explicit, staggered-grid	Requires usage of separate program to calculate geometric elements (GEDA) (14). Designed for river routing, usage for dam-break floods difficult. Requires relatively small time steps and fixed distance steps.
DAMBRK (National Weather Service)	Four-point, weighted-difference, implicit	Designed specifically for dam-break floods, is easily applied and extremely versatile. Handles variable breach and time-rate of breach formation, using storage or dynamic routing in reservoir. Cross sections may be irregularly spaced. Will handle multiple (domino-type) failures. Will route supercritical or subcritical flow.
MOC-LIF (United States Geological Survey)	MOC-explicit method of characteristics. LIF-four- or six-point linear implicit	Not widely used. Treats dam as internal node. Propagates a single bore. Subcritical flow only. Limited to single dams. Requires trapezoidal cross sections, evenly spaced distance steps, small time steps. Relatively expensive computational cost.

convergence is facilitated by the model's ability to route both subcritical and supercritical flows.

In rare cases, when routing through a single downstream channel is judged to be inadequate (i.e., there are very large tributary influences or two major rivers join in the study area), it may be necessary to use a linked one-dimensional routing model. A model recommended for such use is DWOPER (11) developed by the National Weather Service for flood forecasting. It incorporates the same implicit solution techniques as DAMBRK, although it is not designed specifically as a dambreak flood routing model. The author has successfully applied DWOPER to large reservoirs in tandem on the Savannah River.

Determination of Initial and Boundary Conditions for Dam-break Flood Wave Simulation.—The assumptions for reservoir and downstream conditions at time of failure are governed chiefly by study objectives. It is felt that initially the reservoir should be at or near the top of dam. If design data are available, failure should be assumed to occur at maximum design pool level. The downstream valley should be assumed to be in a normal flow condition, with downstream stages determined from steady state backwater calculations. The DAMBRK program will calculate initial backwater profiles prior to performing the dam-break flood routing.

Since the study objectives are to delineate areas always subject to inundation in the event of a dam failure, upstream boundary conditions should represent normal streamflow into the reservoir. Probable maximum or other hypothetical storm inflows would cause the model to represent areas subject to inundation by a large flood added to the dam-break flood. It is the author's opinion that since only 35% of dam failures have been caused directly by spillway inadequacy

TABLE 2.—Selection Matrix

Model (1)	Theoretical applicability (2)	Ease of usage, input data requirements (3)	Versatility (4)	Total usage cost (5)	Availability, documentation, and user experience (6)	Criteria (7)	Total (8)
—	0.30	0.25	0.2	0.15	0.1	Weighting Factor	—
HEC1DB	5	9	6	9	10	—	7.30
USTFLO	7	3	6	6	8	—	5.75
DAMBRK	8	8	9	8	9	—	8.30
MOC-LIF	8	5	5	5	4	—	5.80

(18), areas flooded due to structural failures during normal operational periods constitute basic dam-break flood hazard zones. Inclusion of hypothetical storms in delineation of hazard areas would result in different levels of protection for different dams and downstream areas. Therefore, normal reservoir inflows should be used as upstream boundary conditions. The DAMBRK program will also handle lateral inflows, for which normal values should also be used. If normal values are not known, any flows sufficient to insure model stability may be used up to 10% of the peak flow due to the failure.

The DAMBRK program has the capability to develop its own downstream boundary conditions, so it is only required that the downstream boundary be set at a reasonable distance below the area of interest. In most cases, a final section located 2,000 ft or more below the study area would be satisfactory.

Assumptions Made for Domino-Type Failures.—In domino-type failures, it can be assumed that subsequent earth or rockfill dams will fail as previously described

when they are overtopped by 2 or more ft. Concrete structures can be assumed to fail when overtopped by 30 or more ft, providing they are stable to such an overtopping depth. Combination dams should be assumed to fail when either condition is first met. Most of these assumptions have been proposed by the Corps of Engineers (7). DAMBRK provides for input of tandem reservoir elevations at which failure commences, and, therefore, can easily handle such cases. Bridges can be assumed to wash out immediately or can be treated as dams, depending on the judgment of a qualified hydrologist.

Input Data Requirements for Selected Model.—To simulate a single dam failure using DAMBRK, the following minimum data are required:

1. Breach geometry (simplified to rectangular or trapezoidal shape parameters).
2. Time rate of breach formation.
3. Reservoir elevation-surface area points.
4. Two or three valley cross sections (may be taken from quadrangle sheets if field surveys not available). The cross sections should be chosen to accurately describe valley storage and conveyance. More sections generally improve the accuracy of computed peak stages.
5. Valley roughness coefficients.
6. Boundary Inflow hydrograph and lateral inflow hydrographs.
7. Reservoir elevation at start of failure and initial water surface elevation at downstream end of channel reach—if not known, channel control (normal depth) is assumed.
8. Description on downstream channel flow regimes (subcritical or supercritical).

These data can be assembled using minimal field data surveys, quadrangle maps, and assumptions concerning failure mode. For multiple failures, assumptions for failure initiation as described in the previous section are required.

Summary of Guidelines

1. Recommended Model—DAMBRK (National Weather Service) for most applications.
2. Breach Geometry; (a) Earth or Rockfill Dams: bottom width = $2H$, top width = $4H$; in which H = maximum height of dam; (b) Concrete Dams or Concrete Portions of Dams: total failure; (c) Combination Dams: either (1) or (2), whichever is more likely. If likelihood not known, whichever causes more severe peak stages downstream.
3. Time-Rate of Breach Formation; (a) Earth or rockfill:

$$\left.\begin{aligned} t\,(\text{sec}) &\geq \frac{2.85 \forall_F Tc^{1/2}}{Ac^{3/2}} \\ t\,(\text{sec}) &\leq \frac{1{,}008\,SA}{\bar{B}H} \end{aligned}\right\} \text{Greater Value}$$

(b) concrete:

$$\left.\begin{aligned} t\,(\text{sec}) &\geq \frac{2.58 \Psi_F T c^{1/2}}{A c^{3/2}} \\ t\,(\text{sec}) &\leq \frac{1{,}008\, SA}{\bar{B} H} \end{aligned}\right\} \text{Greater Value}$$

(c) combination, either (1) or (2), whichever is determined to fail.

4. Breach Outflow Discharge Hydrograph—computed by the DAMBRK model using storage or dynamic routing. Normally, storage routing is acceptable for single dams. Domino-type failures would require dynamic routing through downstream reservoirs.

5. Routing Outflow Hydrograph—dynamic routing performed by DAMBRK includes effects of channel storage and conveyance, as well as downstream reservoirs. The model will route subcritical or supercritical flow or two-regime reaches; however, flow regimes may not change during the solution.

6. Initial Conditions—All reservoirs at maximum design pool or top of dam, if design data are not available. Downstream valley profile determined from backwater calculations for normal flow conditions. DAMBRK automatically calculates backwater initial conditions.

7. Boundary Conditions—Main channel and lateral inflow hydrographs to reflect normal flow conditions. Hydrographs may describe steady conditions. Downstream boundary should be placed below area of interest, if it is calculated internally by DAMBRK.

8. Domino-type Failures; (a) Earth or rockfill dams: failure commences when overtopped by 2 or more ft; (b) concrete dams or portions of concrete dams: failure commences when overtopped by 30 or more feet unless known to be unstable at a lower depth of overtopping, in which case the lower depth should be used; (c) combination dams: either (a) or (b), whichever occurs first. Bridges can be assumed to wash out immediately or may be treated as dams, subject to engineering judgment.

9. Input Data Requirements: Minimum data required (for simulation of single dam failure) are: (a) Trapezoidal or rectangular breach geometry; (b) time required for complete breach to form; (c) reservoir elevation—surface area data; (d) downstream cross sections representative of valley storage and conveyance; (e) valley roughness coefficients; (f) upstream boundary and lateral inflow hydrographs (may be steady); (g) reservoir elevation at start of failure and initial water surface elevation at downstream end of channel reach; if not known, channel (normal depth) control is assumed by the model; (h) Downstream channel flow regimes (subcritical or supercritical).

Delineation of Flood Hazard Zones.—Delineation of dam-break flood zones to date has been confined to Gwinnett County, Georgia, where the model flood ordinance will be first adopted. Several dams in Gwinnett County have been analyzed by the Georgia DNR using guidelines previously presented in support of adopted the model ordinance, and the following observations have been made from this experience (23):

1. United States Geological Survey quadrangle sheets or Corps of Engineers flood-plain maps provide adequate horizontal and vertical control. However, specific features, such as bank and invert elevation, must usually be located by field surveys.

2. Field measurements and evaluation of downstream valley characteristics (such as Manning's "*n*") require a skilled hydrologist. Indications are that aerial photo interpretation could provide as efficient means of estimating "*n*" values.

3. A skilled hydrologist is required to run the dam-failure simulation, develop input data, correct errors, and interpret model output. Table 3 details the work sequence and time requirements for specific tasks in analyzing each dam-break flood.

Model Dam-Break Flood Zone Ordinance.—The following amendment to the Gwinnett County Zoning Ordinance has been drafted by the Task Force:

Dam-Break Flood Hazard District.—The Dam-Break Flood hazard District is comprised of those lands that would be subjected to flooding if a structure defined as a Category II dam under the Georgia Safe Dams Act of 1978 should

TABLE 3.—Dam-Break Flood Analysis Tasks

Function (1)	Personnel (2)	Time, in hours (3)
Advance planning and data gathering	1 skilled hydrologist	0.5
Horizontal and vertical takeoffs	1 technician	1.5
Field inspections	1 hydrologist 1 technician	4.0
Data input	1 technician	1.0
Interpretation	1 hydrologist	2.0
Plotting results	1 technician	2.0

fail. The intent of this section is to regulate the use of lands within the dam-break flood zone of Category II dams in order to:

1. Prevent possible loss of human life and damage to persons and property.

2. Prevent Category II dams, as defined under the Georgia Safe Dams Act of 1978, from becoming Category I dams unless such Category II dams meet Category I design criteria.

3. Permit uses that are appropriate in the dam-break flood zone in order to utilize effectively these important lands.

4. Prohibit the construction of commercial establishments, temporary or permanent residences, including camping areas or trailer parks, and other activities in a dam-break flood zone, which would change a dam from a Category II to a Category I dam where loss of human life would occur if the dam should fail unless such Category II dam meets Category I design criteria.

Within the Dam-Break Flood Hazard District, any use is permitted which would not change the category of the dam. Specifically, the following uses are permitted:

1. Agriculture, including forestry and livestock raising, requiring no structures for human habitation within the dam-break flood zone, and including agricultural and forestry access roads.
2. Dams provided they are constructed to meet Category I design standards.
3. Fences.
4. Outdoor advertising signs located not closer than 100 ft to any residential or institutional structure.
5. Roads and parking areas.
6. Public, semipublic, private, and commercial recreation uses requiring no structures within the dam-break flood zone. However, no camping is permitted.
7. Public utility poles, towers, pipe lines, sewage treatment outfalls, and sewage lagoons.

Procedure for Requesting Variation in Dam-Break Flood Hazard Zoning District Boundary Line.—If a property owner can demonstrate to the satisfaction of the Gwinnett County Zoning Board of Appeals through a more accurate engineering survey and flood routing of the dam-break flood by a qualified engineer that his property or a designated portion of it that now lies in the Dam-Break Flood Hazard Zoning District would not cause the dam in question to become a Category I dam, then the Zoning Board of Appeals may change the Flood Hazard District Boundary line accordingly.

The Gwinnett County Zoning Board of Appeals and the property owner involved may seek the advice of the State Soil and Water Conservation Committee in determining the proper District boundary line.

Procedure for Changes in Dam-Break Flood Hazard Zoning District.—If a property owner can demonstrate to the satisfaction of the Gwinnett County Zoning Board of Appeals that the dam above the lands in the Dam-Break Flood Hazard District meets the design criteria for Category I dams as established by the State Environmental Protection Division of the Department of Natural Resources, then the Gwinnett County Zoning Board may at its discretion eliminate or change the boundaries of that district below the dam in question. The Gwinnett County Zoning Board of Appeals and the property owner involved may seek the advice of the State Soil and Water Conservation Committee in this matter.

Conclusions

The development and enforcement of dam-break flood zone ordinances will present many new problems not generally encountered in typical flood-zone regulation. These problems will arise due to inexperience in formulating and interpreting ordinances, nonstandardization of procedures and assumptions used to delineate dam-break flood zones, and conflicting interests in the zoning process. The Georgia Safe Dams Task Force has attempted to adopt workable and realistic standards for determining dam-break flood zones for a large number of dams. The guidelines presented in this paper were developed strictly for the purpose of determining flooded areas, and, while they may be applicable to other types of dam-break studies, all study objectives should be carefully considered before utilizing the recommended methods or assumptions. The Task Force has also sought to minimize the complexity of the model ordinance in order to facilitate its adoption and enforcement. The success or failure of these efforts will become apparent after the adoption of the ordinance by Gwinnett and other counties.

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Appendix II.—Notation

The following symbols are used in this paper:

Ac = breach cross-sectional area at initial water surface elevation (f^2);
$\bar{B}$ = average breach width (f);
Cc = breach celerity at initial water surface elevation (f/s);
g = constant of gravitational acceleration ($32.16\,f/s^2$);
H = total head-on breach at initial water surface elevation (f);
Δh = change in total head on breach during time of breach formation (f);
P = error in peak discharge from that computed assuming instantaneous failure (dimensionless);
Qp = peak outflow from breach (f^3/s);
S = total reservoir storage at time of failure (acre $-f$);
SA = initial surface area of reservoir (acres);
T_c = breach top width at initial water-surface elevation, (f);
t = time from start to completion of failure(s);
$\forall_F$ = volume of material removed from the breach (f^3); and
γ_F = specific weight of fill or dam material (dimensionless). Assumed 2.65 for earth and 2.4 for concrete.

UNSTEADY WATER QUALITY MODEL FOR RIVER NETWORK

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INTRODUCTION

The nature of environmental behavior is complex; superficial analysis of a problem may leave the solution with many uncertainties, thus reducing project reliability. The mathematical modeling approach in water quality river modeling is needed for more precise answers to such problems. If the mathematical model is to be used as a predictive tool in management studies it must accurately reproduce natural processes. Accuracy is based on available data and on model formulation. The uncertainity in the recording and processing of data and the amount of data available are constraints that can make the model unreliable.

The formulation of the hydrodynamic behavior of rivers or estuaries has given good results in the past and has been well defined in many models. The transport processes of a substance in the water body include advection and dispersion. The reaction processes are not well established because they are interdisciplinary. The interaction of the chemical, biological, and physical factors is highly complex and difficult to formulate.

Simulation of hydraulic behavior is achieved through two partial differential equations: the continuity equation, which is based on mass conservation; and the momentum equation, which is based on momentum conservation. Solving these equations requires information on river geomorphology, the values of water discharges or depths at the boundaries, or both, and at the initial time step for all sections. The solution gives the discharge, area, velocity, and water surface level at the sections which provides the basic information for water resource projects, such as flood control, navigation, hydropower, and water quality. The distribution of a substance in a river is simulated by the transport equation which has advection, dispersion, source, and sink terms representing these phenomena. The solution of this equation required knowledge of flow behavior and concentration distribution at the boundaries and at the initial time step.

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Some models are used to couple the three governing equations into two and solve the system of equations. Other models solve the three equations by first solving the hydraulic equations and then the transport equation for as many parameters as may be required. Simulation of a specific substance is related to objectives of the study. It is also a function of a reliable mathematical formulation for the reaction processes and available data. This type of model may include some assumptions that can minimize computational cost and inefficient calculations. Steady flow is an assumption widely used in systems where a critical constant discharge can be assumed in the analysis. In estuaries, rivers near an estuary, or a river where the pollution source is from runoff, the unsteady flow model is a better simulator. In order for the mathematical model to be most exact, it must have a three-dimensional formulation. In practice, this type of model usually requires much computation, storage, and data that are not generally available. Two-dimensional and one-dimensional models can be used for practical purposes. One-dimensional network models are able to simulate accurately a broad, complex system at low cost. The two-dimensional models are commonly used to simulate a specific problem in more detail.

The purpose of this study was to develop a model for unsteady flow conditions that would simulate the hydraulic and water quality response of a one-dimensional river network. Such a model is useful in the simulation of a river near an estuary whose system geomorphology is made up of connected branches and loops. It can also be used with rivers whose islands, tributaries, and meanders are connected with the main flow.

Governing Equations

The gradually varied unsteady flow in a river can be described by two partial differential equations: the continuity equation, which takes into account the continuity of the mass flow, and the momentum equation, which represents the dynamics effects of the flow. The basic assumptions made in the derivation of the equations are: (1) The channel is sufficiently straight and uniform in the reach so that the flow characteristics may be physically represented by a one-dimensional model; (2) hydrostatic pressure prevails at every point in the channel; (3) the water surface slope is small; (4) the density of the sediment-laden water is constant over the cross section; and (5) the resistance coefficient is assumed to be the same as that for steady flow in open channels and can be approximated from resistance equations applicable to open channels or from field survey.

The two basic Saint Venant equations are

$$\frac{\partial Q}{\partial x} + T\frac{\partial y}{\partial t} = q_e \quad \ldots\ldots\ldots\

coefficient; y = the flow depth; g = the gravity acceleration; S_o = the mean bed slope; S_f = the friction slope; and D_l = the dynamic contribution of lateral discharge, given by $(q_e v_e / Ag)$.

The transport of mass in an environment is due to the advection, diffusion, and dispersion processes. The one-dimensional form of the transport equation for a stream has been developed based on some assumptions: (1) A mean value may represent the variation of the concentration and velocity over the cross section; and (2) the longitudinal dispersion coefficient represents the cross product of the longitudinal velocities and concentrations about the cross-section mean plus the spatial mean value of the turbulent diffusivity. The one-dimensional transport equation is

$$\frac{\partial(AC)}{\partial t} + \frac{\partial(QC)}{\partial x} = \frac{\partial}{\partial x}\left(EA\frac{\partial C}{\partial x}\right) + S_l \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which E = the longitudinal dispersion coefficient; S_l = the losses and gains of the system; and C = the substance concentration.

The source and sink term used for biochemical oxygen demand (4) is

$$S_l = -(K_1 + K_3)AC + AL_a + q_e C_{e(\text{bod})} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which K_o = the BOD carbonaceous reaction rate (per day); K_3 = the rate coefficient for the removal of BOD by sedimentation and adsorption (per day); L_a = the rate of addition of BOD along the reach (ppm per day); q_e = the lateral discharge (m^2/s); A = the cross-section area (m^2); and $C_{e(\text{bod})}$ = the concentration of BOD in the lateral flow (ppm).

The source and sink term for dissolved oxygen used (4) is

$$S_l = -K_1 A C_{\text{bod}} + K_2 A (C_s - C) - D_b A + q_e C_{e(\text{do})} \quad \ldots\ldots\ldots\ldots\ldots (5)$$

in which C_{bod} = the BOD concentration (ppm); K_2 = the reaeration coefficient (per day); C_s = the saturation dissolved oxygen concentration (ppm); D_b = the removal of oxygen by benthal deposits and plant respiration and the increase in oxygen through photosynthesis (ppm/day); and $C_{e(\text{do})}$ = the concentration of DO in the lateral flow (ppm).

Numerical Solution

The simulation of the mass transport in a river under unsteady conditions requires the solution of Eqs. 1, 2, and 3. An uncoupled solution method is utilized to solve these three equations. The method solves Eqs. 1 and 2 first, and then in the same time step solves Eq. 3 utilizing unsteady flow conditions determined from the solution of Eqs. 1 and 2. The transport equation is solved for each parameter, first for BOD and later for DO.

Hydraulic Equations.—Eqs. 1 and 2 form a system of nonlinear hyperbolic partial differential equations which can be solved by analytical methods only in special situations. The finite difference methods used to solve those equations are classified into explicit scheme and implicit scheme. The basic difference is that the explicit scheme uses the information from the present time, t, to calculate the variables at the future time, $t + \Delta t$, and, thus, can be solved explicitly. The implicit schemes use the information from the time, t, and t

$+ \Delta t$ to calculate the variables at the time $t + \Delta t$ by the solution of a system of equations. The principal disadvantage of an explicit scheme is the stability requirement, allowing only short time intervals that reduce the efficiency of the computation method. The time step in an implicit scheme is mainly dictated by the accuracy requirements. An implicit scheme is used to solve Eqs. 1 and

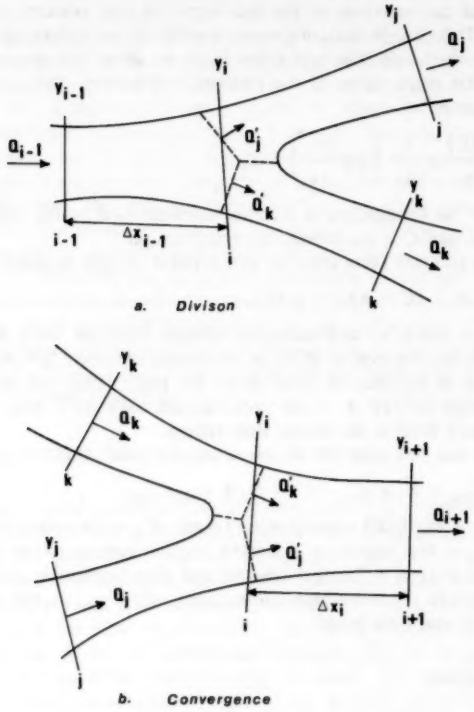

FIG. 1.—Confluence Conditions

2. The scheme is

$$f_{i+1/2} = \frac{1}{2}(f_i^t + f_{i+1}^t)$$

$$\frac{\partial f}{\partial x} \simeq \frac{(f_{i+1}^{t+1} - f_i^{t+1})}{\Delta x}$$

$$\frac{\partial f}{\partial t} \simeq \frac{1}{2\Delta t}[(f_i^{t+1} - f_i^t) + (f_{i+1}^{t+1} - f_{i+1}^t)] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which f = a variable representing Q, y, A, etc.; Δx = the space step; Δt

= the time step; $t + 1$ = the time step $t + \Delta t$; i = the upstream section of the reach; and $i + 1$ = the downstream section. Substituting the numerical scheme in Eq. 6 into Eqs. 1 and 2 for reaches, results in a set of linear equations. Detailed formulation of the system of linear equations is given by Tucci (10). Adding the boundary equations to this set of linear equations, results in the total number of equations equal to the number of unknowns (Q, y, and C at all computation actions). This permits solution of the equations.

In order to proceed with the calculations it is necessary to specify the stages and discharges at all computation sections at the initial time step. Usually these values are not known and should be estimated. A method of estimation is: (1) Interpolate the stages at computation sections and assume constant discharges where there are no confluence nor lateral flows; (2) with the presence of confluences, distribute discharges to the divided channels according to the relative magnitudes of their conveyances; and (3) run the unsteady flow program with constant boundary conditions. Usually after a few time steps a steady condition

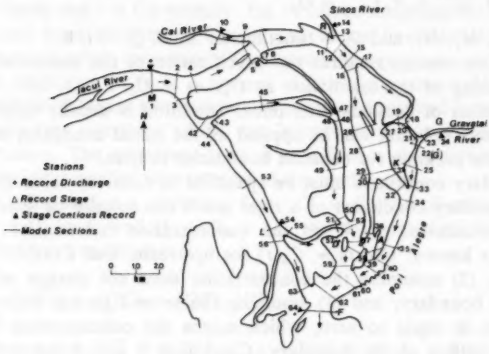

FIG. 2.—Jacui Delta Map

that provides a good initial condition can be obtained. Baltzer and Lai (3) showed the convergence to the same solution using different values of initial discharge. Another procedure to estimate the initial condition is to directly solve the steady backwater equations.

In a river reach with two boundary sections there is an option to specify stages, discharges, or stage-discharge relations as the boundary conditions. When the flow is subcritical, it is necessary to specify one condition at the upstream boundary and the other at the downstream boundary. In this situation one characteristic of the equation propagates downstream and the other propagates upstream. In the case of supercritical flow, both characteristics propagate downstream. Thus, one should specify two conditions at the upstream boundary.

Transport Equation.—Eq. 3 is a parabolic partial differential equation. Numerical solution of this equation here is performed by an implicit backward scheme. This scheme uses the same sections of the hydraulic equations solution. This scheme is

$$f(x,t) \simeq \theta f_i^{t+1} + (1-\theta) f_i^t$$

$$\frac{\partial f}{\partial x} \simeq \theta \frac{(f_i^{t+1} - f_{i-1}^{t+1})}{\Delta x} + (1-\theta) \frac{(f_i^t - f_{i-1}^t)}{\Delta x}$$

$$\frac{\partial^2 f}{\partial x^2} \simeq \frac{1}{\Delta x} \left\{ \theta \left[\frac{(f_{i+1}^{t+1} - f_i^{t+1})}{\Delta x} - \frac{(f_i^{t+1} - f_{i-1}^{t+1})}{\Delta x} \right] \right.$$

$$\left. + (1-\theta) \left[\frac{(f_{i+1}^t - f_i^t)}{\Delta x} - \frac{(f_i^t - f_{i-1}^t)}{\Delta x} \right] \right\}$$

$$\frac{\partial f}{\partial t} \simeq \frac{1}{\Delta t} (f_i^{t+1} - f_i^t) \quad \ldots \ldots \ldots

of the developed model in Jacui Delta as will be mentioned later.

Confluence Equations.—The confluence condition is often treated as an internal boundary according to Chen (5) and Cunge (6). Assuming a steady-state condition at the junction as shown in Fig. 1(*a*), mass conservation is satisfied by the following equation:

$$Q_i = Q_j + Q_k \quad \ldots \ldots \ldots

The advection term in the transport equation, when it is applied for sections k and j in Fig. 1(a), is

$$\frac{\partial(QC)}{\partial x} \simeq \frac{1}{\Delta x}\left[Q_k(C_k - C_i)\right]^{t+\theta} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (12a)$$

$$\frac{\partial(QC)}{\partial x} \simeq \frac{1}{\Delta x}\left[Q_j(C_j - C_i)\right]^{t+\theta} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (12b)$$

The dispersion term is modified when the section is downstream or upstream

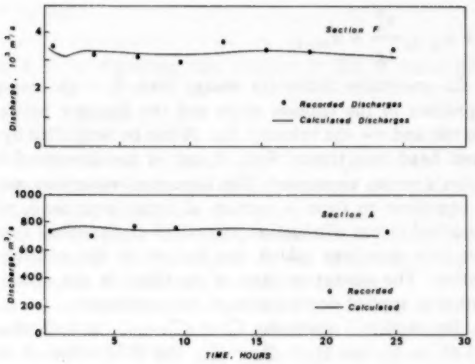

FIG. 5.—Discharges at Sections F and A

of the confluence. The approximation used for the case of Fig. 1(b) at section i is

$$\frac{\partial}{\partial x}\left(EA\frac{\partial C}{\partial x}\right) \simeq \frac{1}{\Delta x^2}\left\{\frac{1}{2}\left[(EA)_i + (EA)_{i+1}\right](C_{i+1} - C_i) - (EA)_k(C_i - C_k)\right.$$

$$\left. - (EA)_j(C_i - C_j)\right\}^{t+\theta} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (13)$$

For section i in Fig. 1(a), the approximation is

$$\frac{\partial}{\partial x}\left(EA\frac{\partial C}{\partial x}\right) \simeq \frac{1}{\Delta x^2}\left\{(EA)_k(C_k - C_i) + (EA)_j(C_j - C_i)\right.$$

$$\left. - \frac{1}{2}\left[(EA)_{i-1} + (EA)_i\right](C_i - C_{i-1})\right\}^{t+\theta} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

Using these approximations in the transport equation for the sections at the confluence results in a numerical equation similar to Eq. 8. Applying the numerical equations for each section inside of the boundaries, results in a set of linear equations. Detailed formulations of these equations are given by Tucci (10).

Solution of the Linear System of Equations.—In matrix notation the system of equations resulting from finite-difference approximation of flow Eqs. 1, 2, 9, and 10, is

$$\mathbf{F} \cdot \mathbf{X} = \mathbf{V} \quad \ldots \ldots \ldots

delta including the Gravatai, the Sinos, the Cai, and the Jacui. The total watershed area at section *F* (Fig. 2) is about 100,000 km^2, which represents one-third of the state.

The Jacui River is the main stream; its watershed at section *M* makes up about 80% of the total watershed. The Jacui Delta is a complex system of branches, confluences, and storage basins, with an area of only 42 km^2. The distance between the confluences is small (small islands), the widths are large (about 1,000 m) in the main channels, and the slope is small. Below the downstream section, the rivers form a series of large lakes that are linked together until they reach the Atlantic Ocean. The delta is about 250 km away from the ocean. On the eastern side of the delta there is a harbor and Porto Alegre, which is the capital of the State, a city of about 1,200,000 people.

The water level in this delta shows a cyclic variation with an amplitude of about 30 cm within a 24-h time period. This cyclic variation is sometimes altered by wind effects and floods. In the dry season when the flow is low, a flow inversion can occur due to the backwater effects from the lakes.

The Gravatai and Sinos Rivers carry domestic and industrial pollution to the delta. In the near future, the Jacui will bring pollution from a petrochemical complex. The Jacui River has a low level of pollution, most of it coming from agricultural sources.

The water from this Delta was used for water supply, waste dilution, navigation, and recreation. Upstream in the Gravatai and Sinos Rivers, the waste is dumped directly into the rivers without treatment; and downstream near the harbor, water is collected for domestic water supply. Water quality is poor during the summer when the flow is low. Due to the complex behavior of this Delta with its flow inversion, it is difficult to decide where the water supply intake should be located. A detailed description of the Jacui Delta, the problems encountered, and the data available for study is provided by Tucci (10).

Available Data.—There are six measuring stations to provide continuous records of stage and other stations for discontinuous readings (Fig. 2). The reference level used was sea level but the reference elevation of some of these stations has not yet been determined. The discharges were recorded in seven sections (Fig. 2). The record of discharge available in those sections is for 24 h (12 noon, April 27, 1977–12 noon, April 28, 1977).

Fig. 2 shows that 64 sections were used in the model to represent the river system. These sections were chosen taking into account the cross-sectional changes and confluence criteria. A complete set of water quality data is not available *th* calibrate the concentration distribution in the mass transport model. The existing data are the concentration of DO and BOD collected at some stations on an occasional basis.

Calibration and Verification of the Hydraulic Parameters.—Data used for the parameter adjustment was from 12 noon, April 27, 1977–12 noon, April 28, 1977. The boundary conditions used in this calibration were the discharges at sections M, J, R, and G, and the stage at section F. The initial conditions for all sections were calculated by assuming an arbitrary initial condition for all sections and running the program, holding the boundary values constant. After about 30 time steps the steady-state condition was reached.

The calibration procedure involves modifying the roughness coefficient (Manning's *n*) at each computational section to simulate stages and discharges of

the main branches using the field data collected at section L (also denoted as section 54), D(57), A(35), and K(29) in Fig. 2. Manning's n in the secondary branches was not adjusted since there were no data available. Detailed descriptions of model calibration and verification are given by Tucci (10).

Measured and calculated values at some sections are shown in Figs. 3–5. The difference between the computed and measured stages was on the order of 1.0 cm in most of the sections. The difference in the discharges was less than 8%. Since the errors in the measurement of these discharges were greater than that, refinement would be a waste of time.

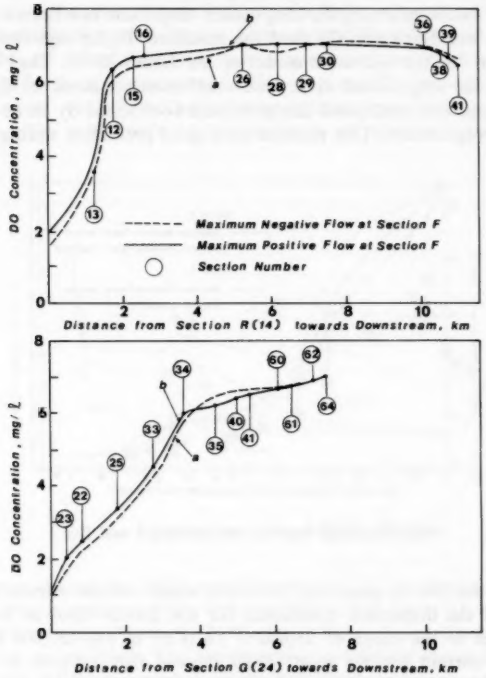
DO Concentration, mg/ℓ
Maximum Negative Flow at Section F
Maximum Positive Flow at Section F
Section Number
Distance from Section R(14) towards Downstream, km
DO Concentration, mg/ℓ
Distance from Section G(24) towards Downstream, km

FIG. 7.—DO Profiles for Option 1

After the parameters were found by model calibration using data collected during a period of 24 h, model verification was required to determine whether the parameters were sufficiently reliable for use in other simulation periods. A 48-h period was chosen for verification when the stage variation was about 0.65 m. In this case it was only possible to verify the stages because the discharges were not available. The configuration, geometry data, parameters, and time step were the same as those used in the model calibration. The boundary conditions used were the levels at sections F and M. The recorded stages used in the

verification were only from sections L and D. The results at sections D and L are shown in Fig. 6. At sections J, R, and G, a constant discharge arriving in the Delta was assumed equal to 30.0 m^3/s. The solution of the mathematical model shows good agreement with the measured values.

Management Use of Model.—The Jacui Delta was used in this study to show the utility of the model in management studies. Assuming the purpose of the study was the evaluation of the water quality level, then the biochemical oxygen demand (BOD) and dissolved oxygen (DO) parameters would be used in performing the analysis. The water quality parameters for the Jacui Delta were estimated in order to perform some management tests.

There is no data concerning the longitudinal dispersion coefficient in the Jacui Delta. Dailey and Harleman (7) used the modified Taylor equation to predict this coefficient for the hydraulic model of the James River. They used a dye test to verify the longitudinal dispersion coefficient predicted by the modified Taylor equation, and multiplied this predicted coefficient by three to account for channel irregularities. This resulted in a good prediction during high-water

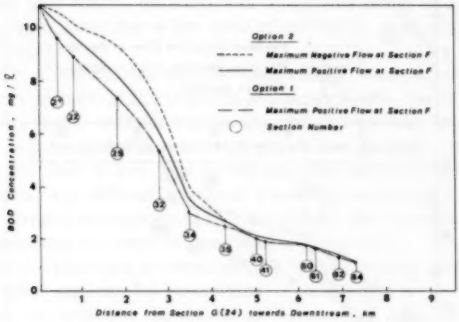

FIG. 8.—BOD Profiles for Options 1 and 2

slack values, but not as good for low-water slack values. Amein and Galler (1) determined the dispersion coefficient for the Lower Chowan River. They obtained values in the range of 25,000 ft^2/h (¢.65 m^2/s)–130,000 ft^2/h (3.36 m^2/s) for no current and no wind. They showed that because of dispersion due to wind and reversed flow, a coefficient of 300,000 ft^2/h (7.8 m^2/s) is typical of most estuary flow. The Jacui Delta is mostly irregular and it has wind and reverse flow effects. Therefore, the longitudinal dispersion coefficient used in this management test was 10 m^2/s. The temperature was 27° C and the decay rate, K_1, was 0.1/day based on data collected in the Sinos River. The saturation concentration C_s is mainly a function of water temperature and atmospheric pressure. Based on the Standard Methods for the Examination of Water and Waste Water (2), this concentration was determined to be 7.8 ppm for a temperature of 27° C. The reaeration coefficient was determined to be 0.5/day by using O'Connor and Dobbins' (8) equations developed for rivers. Due to the lack of data, the values of K_3 and D_b were assumed to

be zero. No sensitivity analysis has been conducted to evaluate the sensitivity of changing water-quality parameters affecting the predicted concentration. However, this can be easily conducted.

Some options were tested with a set of generated boundary conditions. The upstream boundary conditions were assumed constant discharges during the simulation period. The downstream boundary condition at section F was defined by two second-order polynomial equations: one for the positive discharge period, and the other for the negative discharge period. The positive period was assumed to be 16 h and the negative period to be 8 h. The condition used was that the water volume entering the Delta through sections M, J, R, and G should approximately equal the volume leaving the Delta at section F within a 24 h period, assuming the same storage. Using a total flow input entering sections M, J, R, and G, of 150 m^3/s, which is a minimum condition occurring historically, and 390 m^3/s for the maximum positive flow at section F, the resultant maximum negative flow is -105 m^3/s. The following options were tested.

Option 1.—The rivers with the highest levels of pollution were the Gravatai

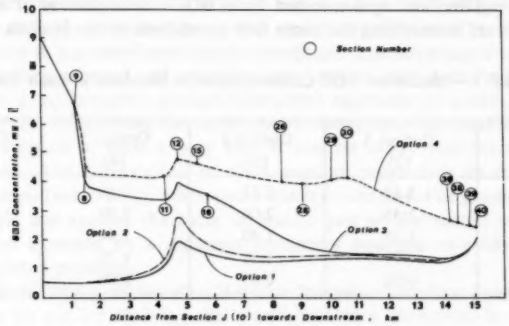

FIG. 9.—BOD Profiles of Four Options Tested

and the Sinos. Based on BOD and DO records, the maximum concentration of BOD at the boundary in the Gravatai River was about 11 mg/L and the minimum DO was about 1 mg/L. In the Sinos River these values were 9 mg/L and 1.5 mg/L, respectively. The concentration at sections M and J was assumed to be 0.5 mg/L for BOD and 7.5 mg/L for DO. Flow conditions at the boundaries are $Q_M = 100$ m^3/s; $Q_J = 10$ m^3/s; $Q_R = 20$ m^3/s; and $Q_G = 20$ m^3/s.

Those values were held constant during a simulation period of 27 h. The results showed that the concentration of BOD was high near the river source and decreased after mixing with Jacui and Cai waters. Fig. 7 shows the profile of concentration along the harbor up to section F. Curve a shows the profile when the flow at section F was the maximum negative flow absolute value. Curve b shows the profile for the maximum negative discharge at section F. It can be seen from the figure that between sections 33 and 34 there was a sudden increase in DO concentration caused by dilution from the confluent river branches. The condition of profile a, as compared to condition of profile

b, decreases DO concentration in the sections near the boundary and increases it downstream.

Option 2.—An alternative option was to test for an increase in flow of the polluted rivers (the Gravatai and the Sinos), and a decrease in flow of the Jacui River, assuming the same concentration values. The flow was Q_M = 70 m^3/s; Q_J = 10 m^3/s; Q_R = 35 m^3/s and Q_G = 35 m^3/s. Fig. 8 shows the BOD profiles according to options 1 and 2. The increase in the profiles of option 2 corresponds to the increase in flow discharge in the Gravatai River from sections G(24)–(34), while the downstream sections caused very little change in the concentration. The sections of branch 48–57 were not affected by this pollution and stayed at low levels (0.5–0.9).

Option 3.—The third option tested for a change in BOD concentration of the Cai River to 9 mg/L and flow conditions at the boundaries to: Q_M = 90 m^3/s; Q_J = 20 m^3/s; Q_R = 20 m^3/s, and Q_G = 20 m^3/s. This condition primarily affected the branch between sections 8–29. There was an increase in BOD concentration before section 28, after which the changes were not significant.

Option 4.—This final option tested for a BOD concentration of 4 mg/L for the Jacui River, maintaining the same flow conditions at the boundary as option 3. The flow of the Jacui River was the largest, and its effects were important in most downstream sections. The concentration profile from section J(10)–section (41) for all four options tested here is shown in Fig. 9.

TABLE 1.—Maximum BOD Concentration at Sections in Each Option

Section (1)	Option 1 (2)	Option 2 (3)	Option 3 (4)	Option 4 (5)
25	8.38	9.53	8.46	8.58
29	1.55	2.04	2.03	4.00
36	1.38	1.40	1.44	3.05
61	1.72	1.80	1.75	2.33
57	0.92	0.95	0.94	2.00

The following is based on two assumptions: that the sources of pollution are restricted to those previously defined, and, secondly, that the generated data are reliable.

The concentration of the rivers entering the Delta mainly affects the concentration of the branches nearby, and usually these values for BOD are high during the period of negative flow. During positive flows, or low tide at the downstream boundary, there is more flow for dilution of the waste, making it the most convenient period for waste disposal. Table 1 lists the maximum concentration value for sections in these options. For instance, the maximum concentration for section 25 was in option 2, where the polluted discharge from the Gravatai was greater. This had a direct effect in this section. Section 57 had the greatest value in option 4 when the Jacui River had the highest concentration. There was not much change in the other tests. For sections near section F and branches 42-53-56 and 48-50-57, there was no significant change in the concentration when the pollution in the Cai, Sinos, and Gravatai Rivers increased. The Cai

River had a major effect on the concentrations of sections between 8–28. The Sinos River primarily affected the sections between 22–35. The Jacui River is the main source of water in this Delta. If water-quality level in the Jacui River decreases, then water-quality level in most of the sections of the Delta will also decrease (Table 1).

Summary and Conclusions

A water-quality mathematical model for a river network under unsteady flow conditions is presented. The model was applied to the Jacui Delta in Brazil. This Delta has the contribution of four rivers with a total watershed of about 100,000 km^2. There are many confluences and storage basins, and there is a harbor located on the eastern side where there is also a large city, Porto Alegre. Two of the rivers are highly polluted, and another one is likely to be polluted in the coming years due to the waste of a petrochemical complex.

The hydraulic part of the model was calibrated and verified utilizing available hydraulic data. However, the parameters of the transport equation for BOD and DO were not adjusted since there was no data. A management option was tested with the model showing the model capabilities. The limitation of the model was in the one-dimensional assumption. The advantages of the model were in handling a broad and complex river system with a minimal computer cost, and in using complete partial differential equations in a one-dimensional level, therefore, minimizing the empirical formulations. The data input to the model is designed in such a way that it can be easily transferred to other river systems. The formulation of the computer program is in module form and can be modified to study other water-quality variables. For example, Simons and Chen (9) has applied the same hydraulic part of the model and replaced the transport equation by a sediment continuity equation to study water and sediment related problems.

It is realized that a model should be calibrated to simulate field conditions in order for its use for studying real world problems. However, in many cases data are not available or are not sufficient for model calibration. A model, if it is based on sound physical processes, can be utilized to evaluate responses of a system to changes and to help decision making.

Acknowledgments

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Appendix II.—Notation

The following symbols are used in this paper:

A = cross-sectional area of channel;
C = substance concentration;
C_{bod} = BOD concentration;
C_{do} = DO concentration;
$C_{e(bod)}$ = concentration of BOD in lateral flow;
$C_{e(do)}$ = concentration of DO in lateral flow;
C_s = saturation dissolved oxygen concentration;
D_b = removal of oxygen by benthal deposits;
D_l = dynamic contribution of lateral discharge;
E = longitudinal dispersion coefficient;
f = function;
g = gravitational acceleration;
h_f = energy head loss;
K_1 = BOD carbonaceous reaction rate;
K_2 = reaeration coefficient;
K_3 = rate coefficient for removal of BOD by sedimentation and adsorption;
L_a = rate of addition of BOD along reach;
Q = flow;
q_e = lateral contribution;
S_f = friction slope;
S_i = source and sink;
S_o = bottom slope;
T = top width;
t = time;
v = mean velocity;
v_e = lateral inflow velocity;
x = distance;

y = water depth;
z = water level;
α_y = correction factor for energy losses;
β = momentum coefficient;
Δt = time increment;
Δx = space increment;
ρ = water density; and
θ = weighting factor of time integration.

WEEKLY MULTIPURPOSE PLANNING MODEL FOR TVA RESERVOIR SYSTEM

By James E. Giles[1] and Walter O. Wunderlich,[2] M. ASCE

INTRODUCTION

The TVA Act of 1933 specifically requires the Tennessee Valley Authority to "regulate the streamflow primarily for the purposes of promoting navigation and controlling floods. So far as may be consistent with such purposes the Board is authorized to provide and operate facilities for the generation of electric energy."

Growth and diversification of water uses and public demand for maintaining a high quality environment have expanded the factors to be considered in reservoir system management. Optimally managing the water resource of a river basin for various uses is a complex, multifaceted problem. As part of a continuing effort to enhance existing multipurpose water resource management methods, a weekly multipurpose planning model was developed to provide a modern analysis tool for effective reservoir system management.

The long-range and day-to-day scheduling problem of a large reservoir is complicated for various reasons. The multiple reservoir system offers a great variety of operating alternatives. Measures for quantitative assessment of operating objectives are difficult to derive but are necessary as decision criteria for selecting between operating alternatives. The future system inputs, such as flows, as well as the system demands, such as power loads are uncertain. Numerous and sometimes conflicting constraints are imposed on releases, elevations, and other system variables. The system characteristics and the operational goals can change with time. Therefore, flexible scheduling and analysis methods are needed which can provide guidance under current conditions or for future situations in which past operation experience is not applicable.

To cope with the increasing complexity of reservoir system management, in 1971 the Tenessee Valley Authority started a program to enhance presently used water management methods. The methods are expected to assist the system manager in finding operation policies which give appropriate consideration to

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the three primary purposes of streamflow regulation (navigation, flood control, and power generation). But they will also support TVA's policy to maintain suitable water quality throughout the valley for various water uses, and to provide water supply and recreation to the extent that this is compatible with the other purposes. The water management methods developed will include quantitative evaluation of all operating purposes so that quantitative trade-offs between operation alternatives can be used in the decision process.

The model described in this paper is a weekly planning model which addresses long-range planning problems using weekly time steps. It was developed for use in system planning and operation studies. Specific applications are: (1) Evaluating the effects of system modifications; (2) evaluating the effects of operation policy changes; (3) studying the effect of changing preferences between water management objectives such as navigation, flood control, power generation, recreation, and water quality management; (4) deriving reservoir operating guides; and (5) examining operation schedules for assumed flow sequences.

Background

In the Tennessee River basin and adjoining Cumberland River basin, TVA owns and operates 28 major hydroprojects. TVA also operates, under contract, six Aluminum Company of America projects and eight Corps of Engineer projects for hydropower generation. In addition, there are several TVA owned multipurpose nonpower projects on small tributaries of the Tennessee River. Water releases at all dams are directed by TVA. Flood control operations of TVA's Kentucky Dam are coordinated with the Corps of Engineers. The Corps is also responsible for flood control operations of the Cumberland River projects.

For the purpose of weekly modeling, the 42 major hydroprojects can be classified according to water storage capabilities into storage reservoirs, prescribed storage reservoirs, and nonstorage reservoirs. Storage reservoirs have storage to annual inflow ratios ranging from 0.2–0.7. These reservoirs are also characterized by considerable water level variations over the year. Norris, Cherokee, Douglas, Fontana and Hiwassee Reservoirs are examples of storage reservoirs. The prescribed storage reservoirs have relatively small volumes compared with annual inflows, and also relatively small water level variations during the year. Their storage to flow ratios range from 0.01–0.08. Reservoirs on the Tennessee River from Fort Loudoun through Kentucky Reservoirs, as well as Barkley and Old Hickory Reservoirs on the Cumberland River, are examples of prescribed storage reservoirs.

The nonstorage reservoirs have negligible storage variations. Cheoah, Calderwood, Chilhowee, Melton Hill, Fort Patrick Henry, Apalachia, the Ocoee Reservoirs, Great Falls, Cordell Hull and Cheatham are nonstorage reservoirs. A schematic representation of the reservoir system and of the various reservoir types is shown in Fig. 1. Details of the operating characteristics of the hydroprojects are given in (7). Because present weekly operations can strongly affect future operations, long-range planning is important to prepare the reservoir system for present and more distant future tasks such as flood control, power generation, low flow augmentation, water quality management, recreation, etc. However, long-range scheduling needs to include only those reservoirs which have storage volume available for this purpose. Of the 42 major hydroprojects in the Tennessee

and Cumberland River basins, 19 reservoirs belong to this category (Fig. 1). These 19 reservoirs contain 60% ($13.4 \cdot 10^6$ acre-ft or 16.5 km^3) of the total useful system storage. The remaining 40% is mostly in the reservoirs on the main stems of the Tennessee and Cumberland Rivers. These reservoirs follow seasonal operating guides rather closely.

Use of a weekly time step in the system model allows considerable reduction in problem size and simulation detail, but requires special modeling assumptions for short-term phenomena. A reduction in problem size is achieved by considering only 19 reservoirs as variables. An example of the reduction in simulation detail is the neglect of travel time for water waves. Since such waves, originating by intermittent operation of hydroprojects, always arrive at the downstream project within the weekly time step, water levels and flows are available at

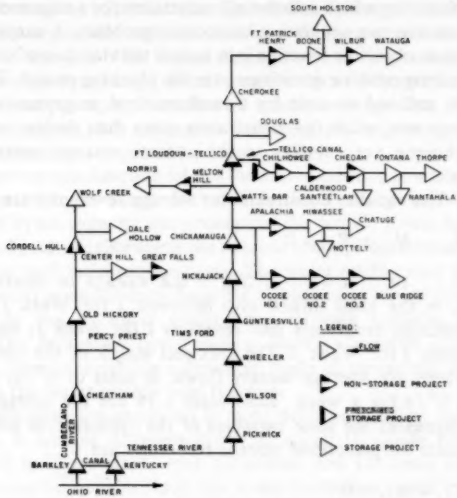

FIG. 1.—TVA-Operated Reservoir System

that project for scheduling in the same time period. However, consideration must be given to short-term phenomena which have a consistent and nonnegligible influence on the computation results. Examples of these include instantaneous flood peaks, steam and hydropeaking, water quality variations in rivers downstream from hydroplants, and pumped-storage operations.

Weekly Planning Model

General Method.—The weekly planning model is designed to find optimal sequences of end-of-week storage levels for 19 storage reservoirs, and weekly releases for 42 reservoirs over a selected planning period. The optimum objective is the minimization of total system operation cost, which includes current cost

over the planning period and expected long-range cost. The total system operation cost is the sum of costs associated with the operation purposes and includes navigation, hydropower generation, flood control, recreation, and water quality costs. Simultaneous consideration of these costs produces a schedule that takes into account the economic merits of all operating purposes. The costs can be multiplied by weights to reflect emphasis given to the operating purposes in addition to their economic value, or to include a safety factor that accounts for approximations in the cost evaluations. The cost minimization is achieved while satisfying all specified constraints. This approach to planning actual operations and to investigating future operation policies is still experimental and does not represent TVA policy at the present time.

Cost minimization proceeds by successively varying an initial set of storage levels (feasible policy) until no further cost reduction within the system is possible. Finding a feasible policy which satisfies all constraints for a sequence of historical inflows can be viewed as a separate minimization problem. A simple but flexible approach to form an objective function is to square the violation of each constraint and sum the resulting positive quantities over the planning period. This objective function is then reduced to zero by a mathematical programming technique. For planning purposes, when few constraints other than design constraints are imposed, a technique very similar to the system cost minimization method following is used.

Constraints.—The equality constraints for storage reservoirs are

$$S_{i,j} - S_{i,j-1} - I_{i,j} - U_{i,j} + R_{i,j} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which $i = 1 \ldots 19$; $j = 1 \ldots J$; $S_{i,j}$ = the storage in reservoir i at end of Week j; $I_{i,j}$ = the local inflow into reservoir i for Week j; $U_{i,j}$ = the release from upstream reservoirs into reservoir i for Week j; and $R_{i,j}$ = the release at reservoir i for Week j. The maximal length of the planning period is $J = 52$. All flows are average weekly flows, in units of ft^3/s, and volumes are in units of ft^3/s for a week. Reservoirs 1–19 are the storage reservoirs. Their storages represent the state variables of the optimization procedure. The equality constraints for prescribed storage reservoirs are

$$-I_{i,j} - U_{i,j} + C_{i,j} + R_{i,j} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which $i = 20 \ldots 42$ and $j = 1 \ldots J$; $C_{i,j}$ = the prescribed storage change in reservoir i (which is not a state variable) for Week j, in terms of average weekly flow, ft^3/s. All other terms are as defined above. The inequality constraints are

$$L_{i,j} \leqq S_{i,j} \leqq V_{i,j} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which $i = 1 \ldots 19$ and $j = 1 \ldots J$; $L_{i,j}$ = the lower storage limit of reservoir i for Week j; and $V_{i,j}$ = the upper storage limit of reservoir i for Week j, all measured in weekly average flow, ft^3/s.

$$M_{i,j} \leqq R_{i,j} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which $i = 1 \ldots 42$; and $j = 1, \ldots J$, in which $M_{i,j}$ = the minimum required release at reservoir i for Week j (ft^3/s).

Objective Function.—The objective function is

$$SF = \sum_{j=m}^{J} F(j) + E(J) \quad \ldots \ldots \ldots

be considered, also known as the "curse of dimensionality." Therefore, after examination of various approaches (4), dynamic programming by successive approximations (DPSA), was used. This method reduces a problem with a high dimensional state space to a sequential, iterative problem with a low dimensional state space (Bellamn, 1957; Larson, 1968; Trott and Yeh, 1973). This earlier study (4) also suggested that DPSA was more appropriate for this scheduling problem than other linear, nonlinear, or dynamic programming methods.

At the start, the DPSA routine requires a feasible set of storages and releases for a specified number of weeks (maximally 52) that satisfies Eqs. 1–4. Dynamic programming is then limited to one reservoir when all other reservoirs are held at their initial levels. As shown in Fig. 2, a corridor of storage levels is established about the given storage levels of the selected reservoir. For any particular week there are three possible states. For each state at time j (end of Week j) there are three possible beginning states at time j-1 (start of Week j). Along the

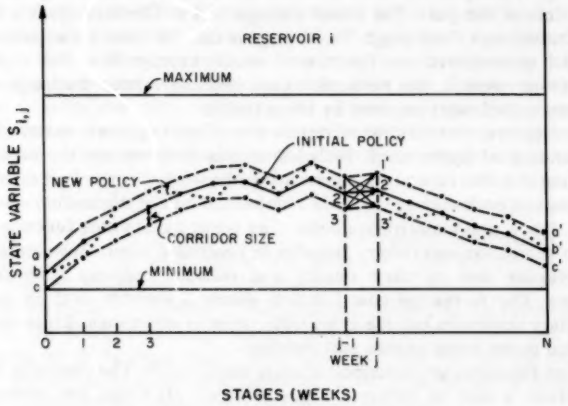

FIG. 2.—Initial Policy and Corridor Used by DPSA

upper path 1–1′ the releases have not changed from those of the initial policy, although the storage levels have been increased by the corridor size. The "best path," that is, the path for which the objective function is minimized, from each of the three states at the end of Week j to a state at the start of Week j, is saved. Therefore, out of nine possible paths, the three best, one for each ending state, are stored. This is done for each stage. In the final week the three objective function values for the three final states consitute the optimal values corresponding to three paths from a beginning storage level to these three final states.

After choosing the best of these three values, a "traceback" is performed which traces the sequence of storage levels that constitutes the least cost path from the final storage level back to the beginning storage level. This new sequence of storage levels, and resulting releases, constitutes the new policy shown in Fig. 2. A corridor is set up around it and another iteration proceeds. When

no further improvement in the objective function is realized, the corridor size is halved and another iteration is made. This procedure continues until the corridor size reaches a specified final size. One "loop" is completed when the above procedure is repeated for each of the 19 storage reservoirs designated as state variables. The method usually converges after two or three loops.

Special Features of Model.—In the weekly planning model, the DPSA procedure has several features designed to reduce computer-storage requirements, to study beginning storage level effects, and to include long-term costs in the objectives.

To reduce the storage requirements, the 19 state variables (reservoirs) are grouped into four subsystems corresponding to tributary basins. A temporary data set, containing power parameters as well as storages and releases, corresponds to each subsystem. The Tennessee main river plants form another subsystem which has a corresponding data set.

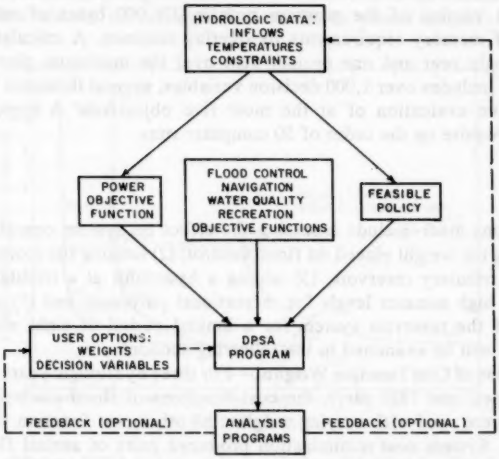

FIG. 3.—Schematic Diagram of Weekly Planning Model

In order to allow the beginning level to vary, the DPSA procedure has an option to include an upper and lower beginning level in the iteration, separated from the given level by the corridor size (Fig. 2). As described in the preceding section, the DPSA procedure can include long-range costs associated with the final storage level of the planning period. For example, if the final storage level is very near its minimum, long-range cost considerations indicate that this would be costly in view of future power generation, recreation, and possibly other requirements, but favorable for flood control. Such a combined cost is represented by a function that assigns a cost to each final storage level, i.e., $E(J)$ in Eq. 5. In principle, this function can be derived by repeated model applications. There are other options built into the procedure which are described in a user guide (6).

The procedures used in the weekly planning model are given in Fig. 3. Satisfying

the constraints over the planning period produces the feasible policy which serves as initial policy. Operation costs and other characteristics of 66 steam units are used to calculate the basic hydropower cost function for a selected project year. Hydrologic information is used to adapt the forecasted normal power loads to the temperature conditions of the selected flow year. The adapted hydropower cost function is then used to calculate total power generation cost as a function of hydro use. Also, the other cost functions are updated for the project year as required. Further input includes weights for the cost functions, starting and ending weeks, reservoirs to be optimized, and hydroplant characteristics. The optimized schedule is then produced by DPSA. The results are processed by a report program, a print plot program, and a graphical display package.

The average CPU time for a 52-week planning period on an IBM 370/165 is 10 min. This time varies with the differences between initial and final policies. The current version of the program runs in 228,000 bytes of memory after reduction of memory requirements by overlay schemes. A calculation run for one hydrologic year and one project year over the maximum planning period of 52 weeks includes over 1,000 decision variables, several thousand constraints, and repetitive evaluation of at the most five objectives. A typical planning study may require on the order of 50 computer runs.

Applications

Applications made include assessing the effect on system operating cost of: (1) Changing the weight placed on flood control; (2) limiting the storage variation in a major tributary reservoir; (3) adding a hydrounit at a tributary dam; (4) maintaining high summer levels for recreational purposes; and (5) planning the operation of the reservoir system for a limited period of eight weeks. These applications will be examined in the following sections.

Modification of Cost Function Weights.—For three hydrologic years, 1975 (wet), 1971 (average), and 1931 (dry), the cost functions of flood control and power generation were used with varying weights; all other cost function weights were set to zero. System cost minimization produced pairs of annual flood control and power costs as shown in Fig. 4. Each curve was obtained for the same initial and final storage levels and a specified hydrologic year, and each curve represents a set of efficient solutions. By moving along the curve, any gain for one operating purpose can only be achieved by some loss for the other operating purpose. The pair of numbers next to the computed points indicates the weight combination for flood control and power.

For example, point (1,0) gives the flood risk cost and power cost if flood control receives a weight of one and power a weight of zero. With this weight combination, the flood risk cost is lowest while the power generation cost is highest. In all years, the same minimum flood risk cost of about $4,000,000 is obtained. This indicates that, with the flood risk function in use even if the reservoirs are operated for flood control exclusively (which keeps them at their lower bounds most of the time), system operation incurs an average annual flood risk cost of about $4,000,000. In other words, the reservoir system cannot provide full flood protection.

A weight on the flood risk function of 10 and a power function weight of

one produces a large reduction of power cost. While it increases the average flood risk cost by about $1,000,000, power cost is reduced by about $100,000,000. As the weight ratio of flood risk cost to power cost is further reduced, some additional reduction in power cost is achieved. However, even with no consideration for flood control (weight ratio = zero), the power cost cannot be reduced below a certain threshold. The curves indicate that power generation is insensitive to flood control for weight ratios from (0, 1) to about (5, 1). This latter ratio could be construed as a lower limit of a possible practical weight ratio.

The effect of varying weights is further illustrated by reservoir water levels for a selected hydrologic year. The levels of a major tributary reservoir for two extreme weight combinations (0, 1) and (1, 0), i.e., no flood control and infinite weight on flood control, respectively, are shown in Fig. 5. In the case of no flood control, i.e., for weights (0, 1), the reservoir levels still exhibit the familiar level rise in spring and drawdown in fall. This level variation indicates how the inflow should be managed for maximum power output. It attempts

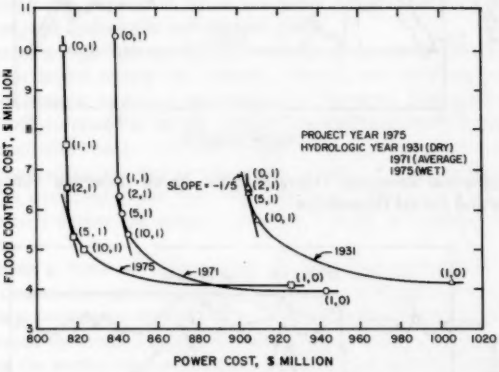

FIG. 4.—Efficient DPSA Solutions Using Various Weights for Flood Control

to avoid water spillage during the flood season, rapidly fills the reservoir after week 10 to provide capacity and energy in storage for future demands, and draws from the reservoir later in the year to meet power loads. In case of exclusive flood control, i.e., for weights (1, 0), the reservoir is operated at very low levels throughout the year because of the varying, but always existing, flood risk cost. A compromise between these extremes, such as the (10, 1) levels, and the actual operation for the historic year, are also shown in Fig. 5.

Reservoir Level Constraint Modification.—A test application was made to investigate the effect of maintaining a major tributary reservoir at high levels throughout the year. A modified minimum reservoir level allowing a storage variation of only about five ft from the top of gates was used. The project year had no unusual features and the hydrologic conditions were average. The results indicated a small reduction in power cost compared with the unmodified (normal) case. But this power cost reduction was counteracted by an increased

flood risk cost that was more than eight times larger than the power cost difference. The total system recreation cost also increased somewhat in the modified case

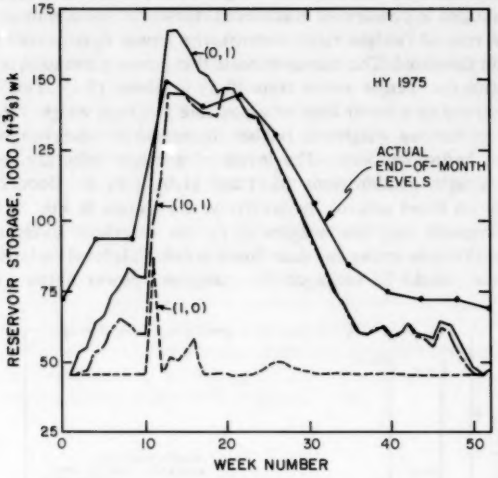

FIG. 5.—Hypothetical Reservoir Operations for Three Objective Function Weight Combinations and Actual Operations

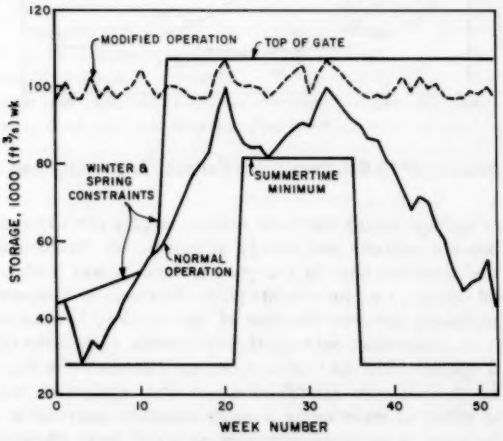

FIG. 6.—Reservoir Operation for Modified and Normal Minimum Level Constraints

because of drawdown in other reservoirs to compensate for the loss in flood control space in this reservoir.

An example of computed reservoir levels is shown in Fig. 6. It can be seen that the modified operation still contributes to flood control and power generation within the limited filling and emptying range. The model can also evaluate the effect of such an operation on other reservoirs in case the system is to make up for this loss in storage space.

Addition of Hydrounit.—The impact of adding a hydrounit to a major tributary project was examined. The evaluation of this capacity change for one project year and one hydrologic year could be accomplished by the model within a few hours. The model results suggested that power cost could be reduced by about $4,000,000 by allowing slightly higher reservoir levels. However, if the unit were used at present reservoir levels without operation policy change, no increase over present flood risk would result, but the power benefits derived from the unit would also be reduced. A small recreation benefit resulted because of the higher storage levels that were scheduled for the increased capacity. These results demonstrate the complex interaction between objectives. They also indicate the type of decisions which the system manager has to make after analyzing the results. A more complete study would, of course, have to include several hydrologic and project years.

Operation with High Summer Levels.—In examining the impact of maintaining high reservoir levels during the summer months, two approaches were used: (1) Fixed constraints; and (2) a recreation cost function. This function attributes a cost to tributary reservoir levels outside a certain indifference range (7).

In the constrained case, reservoir levels in the tributary reservoirs were held above a fixed elevation during the main recreation season (June, July, and August). The hydrologic year used was rather wet overall but was characterized by a dry period during the summer months. The power system had no major unit outages.

To establish a basis of comparison, the reference cases were run without recreation constraints and without the recreation cost function. For the hydrological and power conditions studied, after adding the constraints, power generation costs increased by about $40,000,000 (about 5% of total) over the unconstrained case. Using the recreation cost function instead of rigid constraints would have resulted in a power cost increase of less than half that much. However, using the recreation cost function did not reliably maintain the reservoir levels at or above specified levels. As power costs became high, the levels were drawn down, especially in those reservoirs with low recreation costs. No such trade-off is possible with rigid constraints, which lead to the higher power generation costs.

The reduction in annual hydrogeneration was relatively small in the constrained case. This relatively small difference is contrasted by substantial differences in thermal, purchasing, and peak shaving energy. The hydrogeneration lost while maintaining high levels during the summer period is made up later in the year after the constraints are relaxed. But the thermal generation, peak shaving, and purchasing expenses which occurred during the summer period cannot be compensated for later.

Near Future Scheduling Method.—In planning the operations of the reservoir system the near as well as the distant future must be considered. Otherwise, optimal short-range operation might penalize future operations. The total number of weeks considered constitutes the planning period. The planning period is

subdivided into three segments or time stages: (1) An immediate period (Stage 1); (2) an intermediate period (Stage 2); and (3) a long-range period (Stage 3). These stages are characterized by three types of input data available. For Stage 1, the first week following a given point in time, relatively reliable projected data are available. For Stage 2, several weeks after the first week, less reliable projected data are generally available. But system characteristics and flow sequences of particular interest may be chosen (dry, wet, or average conditions). For Stage 3, in general, only average data based on historical data and expected system conditions are available.

For Stage 3, an operating guide provides a system state versus expected future cost relationship which assigns an expected value to the final system state. The operating guide can be heuristically derived or obtained from the DPSA model (7). The sum of costs over Stages 1 and 2, plus the expected

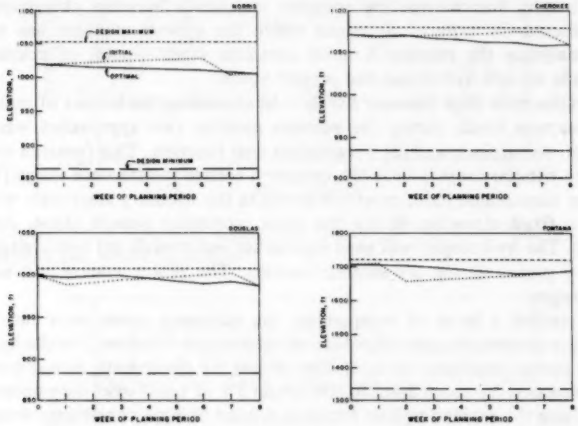

FIG. 7.—Examples of Eight-Week Schedules for Four Major Tributary Reservoirs

future cost associated with the final system state, is minimized by dynamic programming.

In the present application, Week 1 (Stage 1) inflows are based on streamflow forecasts and Stage 2 inflows are selected from the historical record. In this application, in lieu of an operating guide for Stage 3, the end-of-week elevations for the final week of the planning period were fixed as the medians of recent historical operations. A linear programming routine was used to determine an initial sequence of storages and releases which satisfies all operational constraints (3). Then the DPSA algorithm modified this initial sequence in order to minimize the sum of system operating costs for Stages 1 and 2 and the expected power costs for Stage 3. A typical case for an eight-week period is displayed in Fig. 7. The visual shows initial and final (optimal) schedules for four major tributary reservoirs.

The schedules start at given elevations at the beginning of Week 23 (about

June 3), use forecasted flows for Week 23 (the first week of the planning period) and 1971 flows for the weeks thereafter. As soon as new information becomes available, the computation can be repeated. While the level differences between initial policy and the optimized are small in the example shown, the cost differences may be significant. For operational use, only the policy determined for the first week is relevant for possible implementation. Thus, the impact of the assumed endpoint on the first week policy is minor. Presently, neither the fixed endpoint constraint nor the guide developed using the model (7) is considered entirely satisfactory.

In the schedule computed by the model for a planning study, the same importance is given to future weeks' costs as to the first week's cost. For operational use, however, this can attribute too much weight to the uncertain future weeks' inflows. Discounting the future weeks' costs is one method for overcoming this effect. Example runs for a four-week period beginning with Week 8 (about February 23) were made with and without discounting. In the

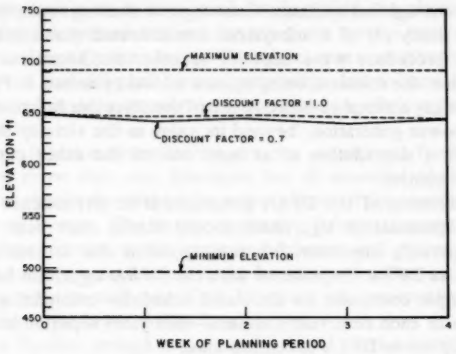

FIG. 8.—Optimal Schedules with Different Discount Factors for a Tributary Reservoir

run with discounting, the ninth week is discounted by a factor of 0.7, the tenth week by 0.7^2 and the eleventh week by 0.7^3. Fig. 8 shows the difference in scheduled elevations for a tributary reservoir. The increased drawdown in the first week, and the reservoir level recovery in the last week, is typical of the differences between the two schedules. The total cost for the discounted run was $340,000 (in nondiscounted dollars) more than for the nondiscounted run. In the discounted run, the model chose a cheaper policy (drawdown) in the first two weeks, and a more expensive policy in the last two weeks. Variable discount factors associated with forecast probability could be chosen. This feature has not been fully explored.

Limitations of Model

An important characteristic of the model is its deterministic nature. A given sequence of inflows is used for a planning period. The DPSA procedure then

iteratively refines an initial schedule until an optimal or near-optimal schedule is found. Thus, full advantage is taken of all inputs over the planning period. The results obtained are at least equal to or better than those produced without this foresight. The effect of foresight on the results increases with the length of the planning period and with the enlargement of the feasible operation space. This effect is generally undesirable and can be alleviated by one of three alternatives: (1) Reducing the planning period; (2) adding constraints; or (3) discounting future costs. In certain planning and operation modes, however, foresight is desirable to determine the best possible operation for an assumed hydrologic and power system scenario.

A limitation of the DPSA procedure is that it does not guarantee a global optimum. If one considers the possible solutions as the surface of a three-dimensional map dotted with peaks and valleys, then the optimal value returned by the procedure would be the lowest point in a valley but not necessarily the lowest point on the map. The complexity of the objective function and the high dimensionality of the computational problem make it very difficult to prove that the solution is a global optimum. Convergence is always monotone, however, and an earlier study (4) of a subsystem demonstrated that solutions obtained by the DPSA procedure were superior to other mathematical programming approaches. Also, the solution, being at least a local optimum, is *Pareto optimal.* This means that any attempt to improve one of the objective function components, for example, power generation, beyond its value in the vicinity of the solution, would result in a degradation of at least one of the other components, for example, flood control.

A second limitation of the DPSA procedure is its dependence on the initial schedule. All optimization algorithms should ideally start near the optimum, and this is especially important for any algorithm that utilizes such extreme decomposition as DPSA. Experience with the DPSA algorithm has shown that satisfying a simple constraint on the initial schedule—minimizing the week-to-week deviation in each reservoir's release—will yield superior initial schedules and significantly lower DPSA execution time.

Another limitation of the model is the use of weekly time steps. Thus, only average conditions for a week can be simulated in the model. For example, an hourly or daily inflow hydrograph for a given week is smoothed when converted to weekly average inflows. However, certain short-time phenomena such as intermittent discharges that have important effects on the results are incorporated by assuming representative hourly patterns within the week. Examples are the power load duration curves, the peak and offpeak patterns of thermal unit operation, and the discharge patterns for water quality (7). Others are incorporated by correlating instantaneous flows to weekly averages or by precomputed look-up tables such as reservoir elevation versus flood damage relationships. A more exact representation of short-time phenomena requires a model with a shorter time step.

Summary and Conclusions

A weekly planning model is described for planning and operational studies of the TVA reservoir system. The model simulates the operation of a multipurpose, multireservoir system by weekly time steps over planning periods of up to

one year. It evaluates an objective function that consists of the weighted sum of five cost functions (navigation, flood control, power generation, recreation, and water quality). Dynamic programming by successive approximations is used to find the storage and release sequences which minimize the objective function over the selected planning period.

For a selected historical year (flows) and a project year (power data and other objective function data), the model produces weekly hydrogeneration, hydrocapacity, releases, end-of-week reservoir levels, and the costs associated with these water levels and releases for all objectives amongst other items of interest. In a typical application, such information is computed for several historical years (for example, dry, average, and wet years) and project years. Constraints, objective function weights or other parameters which are *a priori* unknown, can be determined through sensitivity analysis. This produces the information needed by the system manager to evaluate the cost or other effects of planned system modifications or to develop operation policies. By using all information for the selected planning period, the model can accept guidance by specified constraints or it can search freely within the physical limitations of the system for the optimal solution for any given system scenario.

The model uses weekly time steps. Therefore, transient phenomena such as flood peaks, peak and off-peak power generation, intermittent hydroreleases, etc., are simulated in an approximate way to include their overall impact on the weekly results.

The objective function formulations are not considered final at this time. However, they show that cost functions for all essential objectives can be developed which should represent at least the essence of the operation philosophy for the reservoir system. Due to the extensive information available and the clear relationship between water levels and releases and power generation, at this time only power generation costs can be assessed rather accurately. Other objectives cannot be as well quantified. However, the present functions should serve as paradigms for better future formulations.

The model is flexible enough to permit the use of rigid constraints or penalty functions in place of or in combination with the cost functions. This latter approach is more subjective than the purely economic approach, but is easy to implement and can be used if the economic approach should ultimately prove to be unsatisfactory.

The optimum seeking algorithm used in the model (dynamic programming by successive approximations) cannot guarantee a global optimum, but the solutions found were always better than the initial policies. The optimal solution is characterized by minimal total system cost, as represented by the global objective function. For this solution, all variables are within specified system constraints. Each such solution is an efficient, or noninferior, solution which is characterized by the quality that no objective can be further improved without deteriorating another one. By generating sets of such solutions for various weight combinations, the trade-offs between objectives can be evaluated, and a set of weights can be found that best reflects the intended system operation policy. The optimization method is robust and rather insensitive to the type of objective function formulations used. It can also cope with the complexity of the system without major loss of system simulation accuracy. The repetitive nature of the computations used in finding an optimal solution requires the elimination

of all redundant calculations. This is accomplished in part by using precomputed look-up tables or multiple variate regression functions to represent the cost functions. This may result in some loss of accuracy and application range.

The model does not rely on previous operation experience, but determines reservoir level sequences for 19 storage reservoirs over planning periods of up to 52 weeks, guided solely by the constraints and the objective function. The model has three major limitations. The first is the deterministic character of the model which gives it complete foresight over the inputs. It allows the use of only one prespecified project year and one hydrologic year at a time. The effect of this limitation on the results and how to deal with it has not been fully explored. The second limitation is that a global optimum cannot be guaranteed. The third limitation is that simulating the reservoir system by weekly time steps precludes detailed consideration of hourly phenomena.

The model was tested on various planning problems in which system-wide impacts were of importance. It was found to be a powerful and flexible planning tool that can provide information not obtainable by any other method. The problems addressed typically include over 1,000 variables and several thousand constraints, and demand considerable computer resources. The computer program is efficient, and the time per run, 52 weeks, is modest (10 min on the IBM 370/165). However, the number of parameter combinations to be evaluated in a typical study, combined with the number of hydrologic and project years, is generally large, requiring 50 or more runs per study.

The weekly planning model is not considered a final product. Updating the objective function components as well as modifications of the DPSA procedure will be necessary as system modifications continue and as application experience increases.

The model has been applied in several case studies. The effect of foresight has been found to require very careful interpretation of the model results. The costs produced must be considered as a lower bound on actual costs that can be expected when operating in the face of uncertainty. But information gained by using foresight is useful in the basic explorations of operational and planning alternatives. Measures for limiting foresight include reducing the planning period, use of constraints within the planning period, use of overlapping planning periods, and discounting future costs. A full exploration of the capabilities and limitations of the model has not yet been completed.

Finding operation policies by using cost functions, so far has not been accepted within the TVA organization. Although the objective function approach is generally more flexible than the rigid constraint approach, the latter approach is more common and expedient and has been more acceptable to the decision-maker. As demonstrated herein, however, the model is not limited to the constraint approach and can also be used in an environment where policies are determined based on direct measures for each objective.

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Appendix II.—Notation

The following symbols are used in this paper:

$C_{i,j}$ = prescribed storage change in reservoir i for Week j;
$E(J)$ = expected system cost associated with final Week J system state;
$F(j)$ = weekly sum of weighted cost functions, F_k;
$F_k(j)$ = nonlinear cost function of storages or releases for Week j;
$I_{i,j}$ = weekly local inflow into reservoir i for Week j;
$L_{i,j}$ = lower storage limit of reservoir i for Week j;
$M_{i,j}$ = minimum required release from reservoir i for Week j;
$R_{i,j}$ = release from reservoir i for Week j;
$S_{i,j}$ = storage at reservoir i for end of Week j;
S_n = system state;
SF = sum of weekly cost functions $F(j)$ over time horizon and expected system cost $E(J)$;
$U_{i,j}$ = release from upstream reservoirs into reservoir i for Week j;
$V_{i,j}$ = upper storage limit of reservoir i for Week j;
$\iota - j'$ = possible path in dynamic programming with $\iota = 1, 2, 3$ $j = 1, 2, 3$; and
(W_1, W_2) = relative weight of flood control cost (W_1) versus power cost (W_2).

Optimal Stream-Aquifer Development

By Osman Coskunoglu[1] and C. M. Shetty[2]

Introduction

Efficient and effective planning of water resources for irrigation and hydroelectric generation purposes is a high priority on the economic development agenda in most countries, as food and energy continue to be among the most pressing needs in the world. Motivated by this need, the present study deals with the optimal conjunctive development of surface and ground-water resources for, primarily, irrigation and power generation. Also included are aquifer recharge and downstream water requirements.

The importance of using aquifers in conjunction with the available surface water has been well documented (19,35) and acknowledged internationally (9). Optimal conjunctive development has also been studied extensively, since the optimization model by Buras and Hall (4). Several problems, however, remain to be investigated. Among these are physically justifiable, yet mathematically tractable modelling of the conjunctive development problem; and computationally feasible, yet accurate, solution procedures designed for the mathematical structure of the model. It is the purpose of this paper to further contribute to the mathematical modelling and solution procedure for the optimal conjunctive development problem.

The outline of the paper is as follows. First, the relevant past studies are discussed, concluding with some general observations of the state of the art. This conclusion motivates the next section which includes the specific problem statement. Then the mathematical model for the stated problem is given. The model is essentially a collection of relationships most of which have been previously published. However, the particular synthesis is believed to be a novel one and requires some justification. Thus, following the model presentation, a discussion of the assumptions on which the overall model is constructed is presented in order to justify the particular expressions used in the model. Then the proposed solution procedure, which takes advantage of the mathematical structure of the model, is presented. Rigorous mathematical proofs of the intuitive

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statements presented in the solution procedure are given elsewhere (13). Finally, application of the procedure to a realistic set of data and the numerical results, as well as the computational requirements, are presented.

Relevant Work

The earliest optimization model found in the literature on stream-aquifer development was by Buras and Hall (4). They used dynamic programming for determining the surface storage size and for allocating the available water to surface and groundwater storage. One important result obtained is that if hydropower generation and flood control are not included in the model, the optimal operation of a given system will be that which minimizes pumping cost, while meeting a target demand. Later, Burt (5) obtained a similar result. Buras (3) formulated a dynamic program to determine optimal operating policy of the surface water and groundwater reservoirs with given storage and recharge capacities. Burt (6) also developed a dynamic program by treating the groundwater storage as stock. The first study that uses a set of data from a specific geographic area (San Gabriel Valley in California) was presented by Dracup (15). A parametric linear program was formulated to determine the optimal operating policy for five sources of water—groundwater, local surface water, reclaimed waste water, and imported water from two other regions—to meet the specified requirements of three demand sources—industrial, municipal, and agricultural. Aron (1) expanded on the work of Buras (3) and Dracup (15), and developed a dynamic program to determine optimal water conservation, pumping, import, and distribution policies for the Santa Clara basin in California.

All the studies discussed above attempt to determine the operating policy for surface water and groundwater resources by considering the aquifer as a lump system, neglecting the distributed parameter characteristics of the aquifer system modeled. Models that explicitly couple the distributed parameter system with a management scheme to optimize the conjunctive use of groundwater and surface water have begun to appear within the last decade. Bredehoeft and Young (2) described a simulation model that incorporates the hydrologic interdependencies characteristic of ground water systems. Later, they incorporated this hydrological simulation model with an economic model that represents the response of irrigation water users to variations in water supply and cost (39). This model was the first conjunctive use model that treated the hydrologic interaction phenomenon as an integral part of the modelling process. However, the aquifer behavior was incorporated only through digital simulation and was not directly involved in the optimization. Maddock (23,24,25,26) in a series of papers attempted to eliminate this shortcoming. He first developed an algebraic technological function, which is a Green's function representation, that related seasonal pumping at wells in the system to draw down at those wells. The algebraic technological function allowed an explicit coupling of the groundwater model with a quadratic management model developed (25). Later, the same model was solved to obtain optimal operating rules when the demand and supply sources are stochastic (26). A similar aquifer response approach has been adopted in (16,18,30). The most recent study by Flores, Gutjahr, and Gelhar (17) discussed the benefits and computational costs of distributed parameter models and lumped parameter models. They favored the lumped parameter approach and included

groundwater flow as a stochastic process and aquifer parameters as random variables to obtain a quadratic program with chance constraints. Other studies deal with the optimal conjunctive management problem (8,10,20,32,34,40).

The following three observations can be made: (1) In most of the above studies, irrigation was the sole or main purpose of the conjunctive management, and none of them considered hydroelectric generation as a purpose; (2) most of them, with the exception of Buras (3), and Chaudhry, et al. (1), did not include the crop production function in the objective function; instead, unit cost or unit benefits of water, or both, were considered; and (3) other than Chaudhry, et al. (10), in all the above studies the physical facilities were assumed to be built and operating policy was the only decision to be made. Chaudhry, et al., (10) considered surface water reservoir capacity, conveyance canal capacity, and pumping capacity as decision variables.

Statement of the Problem

In view of the features reviewed above, and reasoning that energy and agriculture are the two more important sectors of our times, we addressed the problem of conjunctive development of two main water resources to serve, mainly, the following important purposes. The components included in the model formulated in this study are as follows: (1) Surface water and ground water as supply sources; (2) irrigation, hydropower, and mandatory downstream requirements as demand sources; (3) surface water reservoir, hydropower plant, canal, wells, and pumps as physical facilities; and (4) sizes of the physical facilities (design) and timing and amount of releases from surface water reservoir and aquifer (operation) as decision elements. A schematic representation of the foregoing is given in Fig. 1.

The problem addressed can be summarized as follows. Determine: (1) Optimal capacities of surface water reservoir, conveyance canals, hydropower plant, and aquifer pumps; and (2) the optimal conjunctive operation of the surface water and ground water resources. The optimization criterion used includes the discounted costs of capital; the operation, maintenance, and replacement expenditures, OMR; and monetary returns from agriculture and energy sales. This criterion is optimized under the physical, hydrological, and demand constraints.

Mathematical Model

Assumptions.—The following four basic assumptions are made. First, the aquifer and its bounding portion of stream are managed by a single agency: a basin authority, of the type suggested by the National Water Commission (12). The agency has complete control of the timing and amount of water to be used from each source and for each purpose. More detailed descriptions of the nature and rights of the agency are presented in Young and Bredehoeft (39) and in Maddock (26). Second, the basin under consideration contains a single stream hydraulically connected to a single aquifer. The interaction between the stream and the aquifer is such that normal seasonal variations in stream head levels to aquifer head levels produce negligible interaction, and interactive flow behavior between the stream and the aquifer is known, independent of

the withdrawals from each. Third, a twelve-month time span can be divided into two periods, based on hydrological characteristics, i.e., dry and wet periods. Fourth, all the physical and economic input parameters are deterministically known or their estimates are available.

Following is the mathematical model. In the next section some implications of the assumptions will be discussed.

Objective Function.—Let f_{1j} and f_{2j} denote the firm (dependable) energy and dump (provided on "if available" basis) energy, respectively, generated in period j with respective prices c_{1j} and c_{2j}. Then the total benefits from the power generated in period $j = R_j^P(f_{1j}, f_{2j}) = c_{1j}f_{1j} + c_{2j}f_{2j}$.

The revenue derived from the allocation of water to a given farm can be

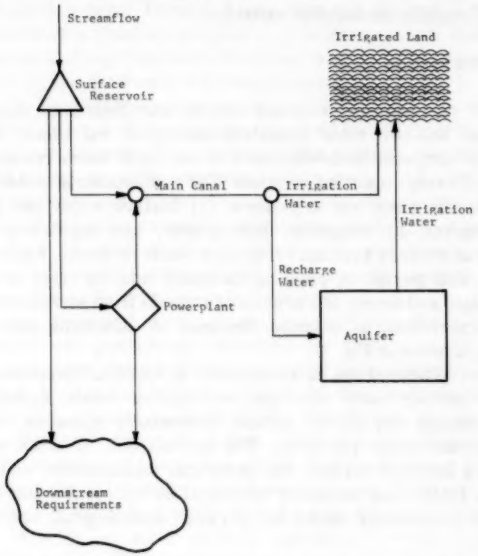

FIG. 1.—Schematic Representation of Water Resources System

obtained if the following information is given: (1) Crop types and pattern in the area; and (2) production function, i.e., a function that relates crop yield to quantities of water supplies for irrigation. It is assumed that the first information is readily available, and obtained by using a method developed for finding optimal cropping pattern (27). The agricultural production function, which has many dimensions, one for each of the variables that affect crop yield, has yet to be derived analytically (11). Cohon (11) also indicated that the most widely used approach for estimating the production function has been empirical investigation. One such investigation indicated that a quadratic function would be a good estimate (3).

In this study, we will assume that the irrigated crop can be categorized into

two groups. Let r_{1j} and r_{2j} be the water received by crop types 1 and 2, respectively, on a farm in period j. Then the total return from the agricultural yield in period j is $R_j^A(r_{1j}, r_{2j}) = d_{cj} - d_{1j}r_{1j}^2 - d_{2j}r_{2j}^2 + d_{3j}r_{1j} + d_{4j}r_{2j}$. In which d_{1j}, d_{2j}, d_{3j}, $d_{4j} > 0$, and d_{cj} are the regression coefficients.

Initial outlays for the dam ($C_r(Q)$), power plant ($C_p(E)$), main canal ($C_c(U)$), and pumps ($C_w(W)$), with respective capacities of Q, E, U, and W, comprise the capital costs: $C(Q, E, U, W)$. The operation, maintenance, and replacement costs are denoted by $C_{rj}(Q)$, $C_{pj}(E)$, $C_{cj}(U)$, $C_{wj}(W)$, and their sum by $C_j(Q, E, U, W)$.

Letting J be the number of periods in a planning horizon, and i be the interest rate, the objective function may be expressed as:

$$v^* = \max \sum_{j=1}^{J} (1+i)^{-j} [R_j^P(f_{1j}, f_{2j}) + R_j^A(r_{1j}, r_{2j}) - C_j(Q, E, U, W)] - C(Q, E, U, W) \quad \ldots (1)$$

Surface Water Constraints.—Let x_{kj} denote the water released from the surface reservoir in period j for the purposes of power generation and meeting mandatory requirements ($k = 1$); power generation and aquifer recharge ($k = 2$); power generation and irrigation ($k = 3$); meeting mandatory requirements only ($k = 4$); aquifer recharge only ($k = 5$); irrigation only ($k = 6$). The mass balance equation for the surface reservoir in period j is

$$\sum_{k=1}^{6} x_{kj} + s_j - s_{j-1} + \epsilon(s_{j-1}, s_j) = q_j \quad \ldots (2)$$

in which, s_j = volume of water in the reservoir at the end of period j; q_j = streamflow in period j. The evaporation losses from the reservoir, $\epsilon(s_{j-1}, s_j)$ is a function of surface area of the reservoir, which is a function of the volume of water in the reservoir (19). The following formula is used to compute the approximate evaporation losses: $\epsilon(s_{j-1}, s_j) = \alpha[(s_{j-1} + s_j)/2]$ in which α_j is a constant that reflects the climatological characteristics of the area in period j.

Stored water and, thus, the reservoir capacity, have a lower bound ($Q_{\min}$) and the latter has an upper bound ($Q_{\max}$)

$$S_j - Q \le 0 \quad \ldots (3)$$

$$S_j \ge Q_{\min} \quad \ldots (4)$$

$$Q_{\min} \le Q \le Q_{\max} \quad \ldots (5)$$

A lower limit on the reservoir capacity and the stored water in the reservoir is imposed to allow for recreational use. The upper limit on the reservoir capacity is necessary either because of financial considerations or the geological conditions of the area, or both.

Groundwater Constraints.—An unconfined aquifer is used for irrigation water in that the aquifer can be replenished from three sources: natural recharges from rainfall, deep percolation of seepage water from conveyance canals and irrigated fields, and artificial recharge from the surface water reservoir. Our lumped parameter model further assumes the following:

1. The saturated thickness of the aquifer is always large compared to that of any drawdown, therefore transmissivity is independent of head. There is no subsidence, and water is instantly released from the aquifer. All wells are fully penetrating. Maddock (23,24) shows that under these conditions a linear ground-water model is applicable.

2. Usage limit of the ground water is based on the "safe yield," in the sense defined in (19), and mining is not allowed.

The equation expressing the mass balance in the aquifer in period j is

$$-L_p(x_{2j} + x_{5j}) - L_r L_1(x_{3j} + x_{6j}) + L_2 x_{7j} - g_{j-1} + g_j = v_j \quad \ldots\ldots\ldots (6)$$

in which x_{7j} = volume of water pumped from the aquifer in period j; g_j = volume of usable water in the aquifer at the end of period j; L_p = fraction of recharge water after losses that percolates to the aquifer; L_r = fraction of irrigation water that percolates to the aquifer; v_j = net inflows to the aquifer from the adjacent aquifers, surface water seepage, and direct rainfall. L_1 and L_2 are factors including farm and canal losses.

The amount of water that can be stored in the aquifer, without causing water logging, is bounded above by H:

$$g_j \le H \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Finally, the amount of water that can be pumped in a period and the pumping capacity are bounded

$$x_{7j} - W \le 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

$$W \le W_{max} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Hydropower Generation Constraints.—The energy balance equation is

$$f_{1j} + f_{2j} - (x_{1j} + x_{2j} + x_{3j}) \cdot \Psi(s_{j-1}, s_j) \cdot \gamma \cdot e = 0 \quad \ldots\ldots\ldots\ldots\ldots (10)$$

in which $\Psi(s_{j-1}, s_j)$ is the average head, on the turbines, of the water in the reservoir, and is computed as a function of the average volume of water in the reservoir in period j. γ and e are unit conversion and efficiency factors, respectively.

The total energy produced is bounded by the power plant capacity

$$f_{1j} + f_{2j} - k_j T_j e E \le 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

in which k_j is the power plant factor and T_j is the number of hours in period j. The firm power is bounded by the demand

$$f_{1j} \le F_j \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

and hydropower plant capacity has an upper limit

$$E \le E_{max} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

Irrigation Constraints.—The following mass balance equation expresses that the amount of water received by the crops, both for consumption and leaching is equal to the amount of the water released from the reservoir and pumped from the aquifer, after losses, plus the rainfall

$$-(1 - L_1)(x_{3j} + x_{6j}) - (1 - L_2) x_{7j} + L_3 r_{1j} + L_4 r_{2j} = p_j \quad \ldots\ldots\ldots (14)$$

in which L_3 and L_4 are leaching factors and p_j is the effective rainfall. The leaching factor is assumed to be constant. The optimal value of this factor can be determined as a function of the amount of irrigation water applied (28).

The total amount of water received by each type of crop cannot be less than the amount of water provided by the effective rainfall:

$$r_{lj} \geq \frac{p_j}{L_3}; \quad l = 1, 2 \quad \ldots \quad (15)$$

Canal Constraints

$$\sum_{k=2}^{6} x_{kj} - U \leq 0 \quad \ldots \quad (16)$$

$$U \leq U_{\max} \quad \ldots \quad (17)$$

Downstream Water Requirements Constraint.—The mandatory release requirements (for flow augmentation, downstream rights, etc.) for each period (M_j) must be satisfied in all periods:

$$x_{1j} + x_{4j} \leq M_j \quad \ldots \quad (18)$$

Nonnegativity Constraints.—Let us define $\mathbf{t}_j = (f_{1j}, f_{2j}, x_{1j}, \ldots, x_{7j}, r_{1j}, r_{2j})$ as the operation variables vector and $\mathbf{d} = (Q, E, U, W)$ as the design variables vector. Then,

$$\mathbf{t}_j, s_j, g_j, \mathbf{d} \geq 0 \quad \ldots \quad (19)$$

Summary.—The optimal conjunctive development problem can now be summarized as: Determine $\mathbf{t}_j$, s_j, g_j ($j = 1, \ldots, J$) and $\mathbf{d}$ that maximize Eq. 1 subject to (for $j = 1, \ldots, J$) Eqs. 2–19.

Discussion of the Model

In developing the above model we have attempted to keep a balance between a truly representational, and necessarily more complex model, and the associated difficulty in analyzing the model (Fig. 1). We have considered a recent study by Rogers (33), which essentially rejected the hypotheses that for water resources systems, less complex models are a more appropriate choice than the more complex models. Two irrigation examples he analyzed, however, raised the possibility that the errors associated with the choice of the simpler models do lie within the range of errors that one might expect due to the estimation of parameters in the more complex models.

At this point, it may be worthwhile to discuss in detail three major assumptions on which our optimization model is based: (1) Parameters are deterministically known; (2) the aquifer is a lumped parameter system; and (3) operation costs of the physical facilities are functions of their capacities, rather than their operation level.

Stochastic effects have been considered in several conjunctive management studies. Random recharge or streamflow has been incorporated in some of the models (5,6,7). Maddock (26) has also considered the effects of random demand. Recently, Flores, Gutjahr, and Gelhar (17) extended these earlier works, and

treated the groundwater flow as a stochastic process and the aquifer parameters as random variables. The conclusions of their analysis indicated that stochastic effects are not very important in arriving at an operating policy but are important in determining the expected cost. Maddock (26) also noted that the operating rules are insensitive to demand variance. These conclusions imply that the optimal operating policy determined by the deterministic model given in this study is satisfactory. The same conclusion, however, cannot be stated for the optimal design obtained. It is not clear how the variance in the optimal cost and return will change as the values of design variables varied. If this change is not significant, then the optimal net return corresponding to each possible design combination will be affected similarly; therefore, the optimal design found for the deterministic case will be the same as for the stochastic case, albeit difference in net return is possible. This aspect needs further investigation.

In Haimes and Dreizin (18), it was claimed, without any supporting results, that considering an aquifer as a lump parameter system, rather than a distributed parameter system, would introduce considerable error. Flores, Gutjahr, and Gelhar (17), on the other hand, examined a lumped parameter system. In order to evaluate the limitations of their model in a management context, they compared their numerical results with results from Maddock (26), who used a distributed representation of the aquifer. It was found that there are significant differences in the stream-aquifer flow and these lead to a slight difference in the objective function. The computational burden, on the other hand, increased drastically in the case of distributed parameter representation of an aquifer. Maddock (24) indicated that the computational cost might well exceed the cost of drilling and completing a well. In view of these conclusions, we believe that the lump parameter representation in this study is justifiable.

The third main assumption is that the OMR costs of each physical facility is considered as a function of its capacity rather than the operation level of that facility. This assumption is generally accepted as reasonable for reservoirs, canals, and powerplants (9,22,29). The operation cost of a pump, however, highly depends on the level of operation, as the amount of power used will depend on the amount of the water pumped and the drawdown. The drawdown will not change significantly since in this study it is also assumed that the safe yield policy is used in the aquifer. The amount of water pumped, however, will not necessarily equal the capacity at all times. Thus, an overestimation of the operation costs will be introduced. This problem can be overcome by the following iteration. First, the operation costs for the pumps can be taken as a function of their capacities. In the second iteration, the operation cost can be taken as a function of the amount of pumpage obtained in the first iteration, and so on. This procedure is not followed in this study.

Solution Procedure

The mathematical programming model given by Eqs. 1–19 has the following salient features: (1) Large scale, e.g., if the planning horizon is taken as 100 periods then the model has 1,304 variables and 1,303 constraints, excluding nonnegativities and initial conditions; (2) the objective function is nonconvex, because R_j^A is concave (3,21), whereas $-C_j$ and $-C$ are convex (22,29). It is also separable and contains only a few of the variables of the problem;

(3) the feasible region defined by Eqs. 2–19 is not convex because of the nonlinear equality constraint given by Eq. 10; and (4) the constraint matrix has a linked structure with coupling variables.

Using a standard mathematical programming algorithm to solve the model is a formidable task because of the size and nonconvex nonlinearities. Therefore, in this study we developed a solution strategy based on two concepts. First, it is generally computationally more efficient to solve a number of smaller problems several times than it is to solve a large, complex problem once. Second, by using empirical insights into the physical system some simplifications can be achieved in solving the decomposed problem.

Before examining the solution procedure, we will rewrite the model given by Eqs. 1–19 in a compact form. Let

$$\phi_1(\mathbf{d}) = \sum_{j=1}^{J} (1+i)^{-j} C_j(Q, E, U, W) + C(Q, E, U, W)$$

$$R_j(\mathbf{t}_j) = (1+i)^{-j} [R_j^P(f_{1j}, f_{2j}) + R_j^A(r_{1j}, r_{2j})] \; j = 1, \ldots, J$$

$$\chi = [(\mathbf{d}, \mathbf{t}, \mathbf{s}, \mathbf{g}): \quad \text{Eqs. 2–19} \quad \text{are consistent for} \quad j = 1, \ldots, J]$$

in which $\mathbf{t} = (\mathbf{t}_1, \ldots, \mathbf{t}_J)$, $\mathbf{s} = (s_1, \ldots, s_J)$, and $\mathbf{g} = (g_1, \ldots, g_J)$. Also let $v(\mathbf{d}, \mathbf{t})$ denote the objective function given by Eq. 1. Then the compact representation of Eqs. 1–19 will be:

$$\text{P: } v^* = \max \{v(\mathbf{d}, \mathbf{t}): \quad (\mathbf{d}, \mathbf{t}, \mathbf{s}, \mathbf{g}) \in \chi\}$$

Decomposition.—Problem P may be decomposed as the design and operation problems such that $\mathbf{d}$ will be the variable vector of the former and $(\mathbf{t}, \mathbf{s}, \mathbf{g})$ will comprise the variables of the latter. Mathematically, this can be achieved as follows:

$$\max_{(\mathbf{d},\mathbf{t},\mathbf{s},\mathbf{g}) \in \chi} v(\mathbf{d}, \mathbf{t}) = \max_{(\mathbf{d},\mathbf{t},\mathbf{s},\mathbf{g}) \in \chi} \sum_{j=1}^{J} [R_j(\mathbf{t}_j) - \phi_1(\mathbf{d})]$$

$$= \max_{\mathbf{d} \in D} \left[\max_{(\mathbf{t},\mathbf{s},\mathbf{g}) \in T(\mathbf{d})} \sum_{j=1}^{J} R_j(\mathbf{t}_j) - \phi_1(\mathbf{d}) \right]$$

$$\max_{\mathbf{d} \in D} [\phi_2(\mathbf{d}) - \phi_1(\mathbf{d})] \quad \ldots \quad (20)$$

in which $D = \{\mathbf{d} \geq 0$: Eqs. 5, 9, 13, and 18 are consistent$\}$, $T(\mathbf{d}) = \{(\mathbf{t}, \mathbf{s}, \mathbf{g}) \geq 0$: Eqs. 2–4, 6–8, 10–12, 14–17, and 19 are consistent for $j = 1, \ldots, J$ and for a given $\mathbf{d} \in D\}$.

If the operation problem defined as

$$\text{OP: } \phi_2(\mathbf{d}) = \max_{(\mathbf{t},\mathbf{s},\mathbf{g}) \in T(\mathbf{d})} \sum_{j=1}^{J} R_j(\mathbf{t}_j) \quad \ldots \quad (21)$$

can be solved, then the design problem given by Eq. 20 is a simple problem as $\mathbf{d}$ contains only four variables (Q, E, U, W) and D has only four constraints in the form of upper and lower bounds.

Operation Problem.—The structure of the Problem OP (separable and multistage) lends itself naturally to be solved using dynamic programming. Let

$V_j(s_j, g_j)$ denote the optimal discounted revenue from stages 1 through j, if the terminal values of the state variables are s_j and g_j ($j = 1, \ldots, J$). The forward recursive equation is as follows:

$$V_j(s_j, g_j) = \max_{\substack{t_j \in X_j \\ s_{j-1} \in S, g_{j-1} \in G}} [R_j(t_j) + V_{j-1}(s_{j-1}, g_{j-1})]$$

in which, $X_j = [t_j : (t_j, s_{j-1}, s_j, g_{j-1}, g_j) \geq 0$ satisfy Eqs. 2, 6, 8, 10–12, 14–17, and 19]

$S = (s_{j-1} : Q_{\min} \leq s_{j-1} \leq Q)$

$G = (g_{j-1} : 0 \leq g_{j-1} \leq G)$.

Note that the Cartesian product of sets $X_1, \ldots, X_j$, S, and G yields $T(d)$.

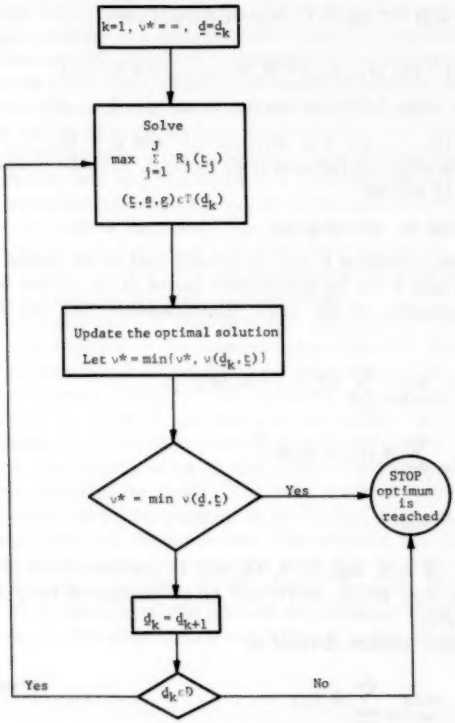

FIG. 2.—Solution Strategy for Problem *P*

Also note that, strictly speaking, X_j is a function of $(s_{j-1}, s_j, g_{j-1}, g_j)$ since it includes Eq. 10 as well as the state transformation functions, i.e., Eqs. 2 and 6. Now, solving the above dynamic programming problem is not easy,

for two reasons. First, there is no possibility of obtaining a closed form solution for $V_j(s_j, g_j)$. Therefore, a discretization on the state space and, hence, on the feasible decision space (X_j) is necessary. Since the vector $\mathbf{t}_j$ contains 11 decision variables, this discretization would be computationally prohibitive. The second reason arises from the nonconvexity of the set X_j, caused by Eq. 10. To alleviate these problems, we may "project" the recursive equation onto the state space as follows (recall that the feasible decision set should actually be written as $X_j(s_{j-1}, s_j, g_{j-1}, g_j)$, but for abbreviation purposes the argument will be omitted):

$$V_j(s_j, g_j) = \max_{\substack{s_{j-1} \in S \\ g_{j-1} \in G}} \max_{t_j \in X_j} [R_j(\mathbf{t}_j) + V_{j-1}(s_{j-1}, g_{j-1})]$$

$$= \max_{\substack{s_{j-1} \in S \\ g_{j-1} \in G}} [\max_{t_j \in X_j} R_j(\mathbf{t}_j) + V_{j-1}(s_{j-1}, g_{j-1})]$$

$$= \max_{\substack{s_{j-1} \in S \\ g_{j-1} \in G}} [R_j^*(s_{j-1}, s_j, g_{j-1}, g_j) + V_{j-1}(s_{j-1}, g_{j-1})] \quad \ldots\ldots\ldots\ldots \quad (22)$$

in which $R_j^*(s_{j-1}, s_j, g_{j-1}, g_j) = \max [R_j(\mathbf{t}_j)\colon \mathbf{t}_j \in X_j(s_{j-1}, s_j, g_{j-1}, g_j)]$ will be

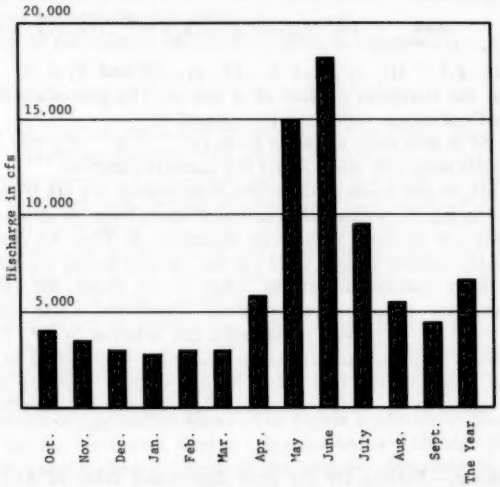

FIG. 3.—Discharge at a Key Gauging Station on Snake River Basin

referred to as the Stage Problem (SP), which solves the following problem: given the water levels at the beginning and end of a stage (period), what is the optimal return from that stage? The significance of the above "projection" procedure may become more apparent noting the following:

1. Given R_j^*, solution of Eq. 22, by state discretization, is simple.
2. Solution of Eq. 22 can further be simplified by noting that for some s_{j-1},

TABLE 1.—Results of

Grid Points				
Run (1)	Reservoir capacity (Q, 20^3 acre-feet) (2)	Powerplant capacity (E, megawatts) (3)	Canal capacity (U, 10^3 acre-feet) (4)	Pumping capacity (W, 10^3 acre-feet) (5)
One	600,900,1200	50,75,100	700,1000,1300	400,600,800
Two	750,900,1050	60,75,90	1100,1300,1500	300,400,500
Three	675,750,825	70,75,80	1350,1500,1650	350,400,450
Four	725,750,775	65,70,75	1600,1650,1700	375,400,425
Five	740,750,760	65,70,75	1680,1700,1720	365,375,385

[a] Q = reservoir capacity in thousands of acre-feet; E = power plant capacity in megawatts; acre-feet.

$s_j \epsilon S$ and g_{j-1}, $g_j \epsilon G$, we may have $X_j(s_{j-1}, s_j, g_{j-1}, g_j) = \phi$. Thus Eq. 22 can be written as

$$V_j(s_j, g_j) = \max_{(s_{j-1}, g_{j-1}) \epsilon \Omega(s_j, g_j)} [R_j^*(s_{j-1}, s_j, g_{j-1}, g_j) + V_{j-1}(s_{j-1}, g_{j-1})] \quad (23)$$

in which, $\Omega(s_j, g_j) = \{(s_{j-1}, g_{j-1})$: $s_{j-1} \epsilon S$, $g_{j-1} \epsilon G$ and $X_j \neq \phi\}$. Clearly, Ω is a subset of the Cartesian product of S and G. The procedure of obtaining Ω is given by Coskunoglu with a complete proof (13).

3. Solving SP is also easy, since for given $(\bar{s}_{j-1}, \bar{s}_j, \bar{g}_{j-1}, \bar{g}_j)$, the $\bar{X}_j$ contains linear constraints only, and since $R_j(\mathbf{t}_j)$ is a concave function.

4. Recall that, at this phase of the solution procedure, we are trying to obtain $\phi_2(\mathbf{d})$, defined by Eq. 21, which is the maximum return from the optimal operation of a set of physical facilities with given capacities, $\mathbf{d}$. Then we can solve Eq. 20 and obtain the optimal design. Only for this optimal design we are interested in how to actually operate the system. Thus, at this phase, the values of the operation variables, $\mathbf{t}$, are of no interest. Therefore, some mathematical manipulations can be employed to expedite the solution of SP. Specifically, it was possible (13) to use constraints and variables aggregation to obtain the exact optimal objective function value for SP without explicity determining the optimal values of $\mathbf{t}_j$. This refinement decreased the computational time significantly.

Design Problem.—Solving OP for each discretized value of $\mathbf{d} \epsilon D$ will yield $\phi_2(\mathbf{d})$. Solution of (20) to determine the optimal design, $\mathbf{d}^*$, is now trivial. However, solving (OP) for each discrete $\mathbf{d} \epsilon D$ values may require excessive computation. It can be shown, however, that there exists $D' \subset D$ such that $\max_{\mathbf{d} \epsilon D} [\phi_2(d) - \phi_1(d) = \max_{\mathbf{d} \epsilon D'} [\phi_2(d) - \phi_1(d)]$. Complete proof of above equation and the details of obtaining D' is given in (13). The insights into the physical system, which are utilized in determining D', are, first, for small reservoir capacities (Q), it will be nonoptimal to have large powerplant (E) and large main pipe (U) capacities. The second consideration to reduce the admissible region of $\mathbf{d}$ is the following. Suppose for a fixed $\mathbf{d} = \bar{\mathbf{d}}$ the solution to OP

Computer Runs

Solution				Optimal return ($10^6) (10)	Execution time (CP seconds) (11)
Q^* (6)	E^* (7)	U^* (8)	W^* (9)		
900	75	1300	400	699.799	1339.928
750	75	1500	400	705.157	1152.356
750	70	1650	400	707.192	567.916
750	70	1700	375	707.557	563.849
750	70	1700	375	707.557	631.232

U = canal capacity in thousands of acre-feet; and W = pumping capacity in thousands

is obtained. If any one or more of the facilities are not used to its full capacity, then any slack capacity can be reduced from $\tilde{\mathbf{d}}$ without influencing the optimal gross revenues but reducing the costs.

A summary of the solution procedure is given in Fig. 2.

Application

To apply the solution procedure developed in this study, the Upper Snake River Basin in Idaho was selected. The reasons for this selection are (31,36,37,38): (1) Hydropower generation potentials are high in the area; (2) this is a highly agricultural area with considerable demand for irrigation water; (3) an aquifer exists near the river basin; (4) most of the data needed for the solution of the model developed in this study are available in this area; (5) to these investigators' knowledge no other work that addresses the problems in this present study has been done for this basin.

Our intention is not to analyze the Upper Snake River Basin but rather to demonstrate the application of the model and the solution procedure. In view of this, a brief description of the application area and the input data is given below. Further details of the data used are given in Coskunoglu (13).

Characteristics of Application Area.—The Upper Snake River Basin is located within the Pacific Northwest Region (also known as Columbia-North Pacific Region). The general characteristics of the area are described as follows: the region's economy is based on its land, mineral, and water resources (27). Fish and wildlife resources and the abundant recreation opportunities are also important. Although the water supply in total is abundant, spatial distribution varies widely and seasonal flows are low in many of the smaller streams. In this region, irrigation uses a significant amount of water. Another large user is the steam-electric powerplants. According to a recent report, hydropower potentials of the region are also the highest in the nation (38).

The Snake River is a major water resource in this area and presently is only partially developed for irrigation, hydropower generation, flood control, and recreation purposes. On the Snake River about 55 mi southeast of Idaho

Falls, Idaho, Palisades Dam is a large zoned earthfill structure. At the time of construction (completed in 1957), this was the largest earthfill dam built by the Bureau of Reclamation. The reservoir supplies supplemental irrigation water to some 650,000 ares of land. The Palisades Powerplant has a total capacity of 114,000 kW with an estimated future average annual output of 635,000,000 kW h (36). The hydrological characteristics of the area in which the Palisades Reservoir is located is summarized in Fig. 3.

The Snake River Basin is rich in easily accessible groundwater. The basin is divided into 19 groundwater subareas (31). For the purpose of this study only the Rockland Valley-Michaud Flats district will be considered. Hydrological and geological details of the area are given (31). Water level measurements

TABLE 2.—Optimal Net Return in Million Dollars for Each Design Combination in Computer Run 5

Q^a \ E^a (1)	65 (2)	70 (3)	75 (4)	65 (5)	70 (6)	75 (7)	65 (8)	70 (9)	75 (10)
				(*a*) $W^a = 365$					
	$U^a = 1680$			$U^a = 1700$			$U^a = 1720$		
740	691.275	691.526	691.033	691.418	691.670	691.176	691.447	691.699	691.205
750	706.745	707.050	706.568	706.727	707.033	706.549	706.595	706.899	706.416
760	705.978	706.269	705.775	705.960	706.250	705.754	705.825	706.115	705.619
				(*b*) $W^a = 375$					
	$U^a = 1680$			$U^a = 1700$			$U^a = 1720$		
740	691.239	691.491	690.997	691.407	691.659	691.165	691.497	691.749	691.255
750	707.241	707.546	707.065	707.252	707.557	707.073	707.181	707.484	707.001
760	706.474	706.765	706.271	706.484	706.773	706.278	706.411	706.700	706.203
				(*c*) $W^a = 385$					
	$U^a = 1680$			$U^a = 1700$			$U^a = 1720$		
740	790.952	691.204	690.710	691.120	691.372	690.878	691.211	691.462	690.968
750	706.953	707.258	706.775	706.963	707.268	706.785	706.893	707.195	706.712
760	706.185	706.475	705.981	706.195	706.484	705.989	706.121	706.411	705.914

[a] Q = reservoir capacity in thousands of acre-feet; E = power plant capacity in megawatts; U = canal capacity in thousands of acre-feet; and W = pumping capacity in thousands acre-feet.

in observation wells show that the water table has declined. Rough estimates indicate that total annual groundwater outflow from the valley is about 50,000 acre-ft. Furthermore, water is also withdrawn from the aquifer for domestic and industrial uses. However, the main concern in the area is not the depletion of the groundwater, but water-logging.

Input Data.—Most of the hydrological data needed for this study were obtained through: Bureau of Reclamation (now, U.S. Department of the Interior, The Water and Power Resources Service) reports; U.S. Geological Survey reports; U.S. Water Resources Council publications; reports obtained from the Bureau of Reclamation staff in Idaho; and personal communication with the Bureau of Reclamation staff in Idaho. The economic data was obtained from other

published research (3,14,22). These references provide the return and cost functions used in this study.

Those input data that could not be obtained in the sources mentioned above are handled in two ways. They are either arbitrarily assumed or picked from general reference texts (21). Included in the former group are rate of increase in the prices of electrical energy and agricultural products, initial storage in the reservoir and aquifer, and downstream water requirements.

Computational Procedure and Results

In the model formulation we assumed that a 12-month time span can be divided into two periods, based on hydrological characteristics, i.e., dry and wet periods. For the application area considered, the wet period is October–March, and the dry period is April–September (Fig. 3).

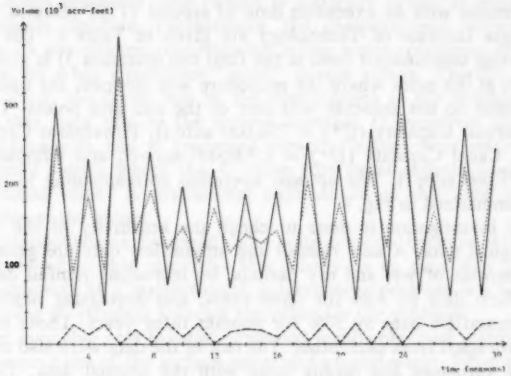

FIG. 4.—Optimal Releases from Surface Water Reservoir (· · ·) and from Aquifer (- - -) as Streamflow (——) Fluctuates over Time

Since continuous hydrological data was available for only 15 yr (30 periods), the following approach was adopted to extend the results to the standard 50-yr planning horizon. Let $v_{50}(v_{15})$ denote the net present worth of the 50 yr (15-yr) of operation less the initial investment. Also, let $\bar{\mathbf{t}}$ and $\bar{\mathbf{d}}$ be the solutions of the problem for $J = 15$, then

$$v_{15} = \sum_{j=1}^{15} (1+i)^{-j} [R_j(\bar{t}_j) - C_j(\bar{\mathbf{d}})] - C(\bar{\mathbf{d}})$$

Let $P_j = R_j(\bar{t}_j) - C_j(\bar{\mathbf{d}})$, and also let $\bar{P}$ be the average annual revenue that is

$$\bar{P} = \left[\sum_{j=1}^{15} (1+i)^{-j} P_j\right] \Big/ \left[\sum_{j=1}^{15} (1+i)^{-j}\right]$$

$$\text{then,} \quad v_{50} = \bar{P} \sum_{j=1}^{50} (1+i)^{-j} - C(\bar{\mathbf{d}})$$

yields the objective function value for 50 yr planning horizons.

The solution procedure developed in this study solves the overall design and operation problem by obtaining the optimal operation of the reservoir and aquifer, and the return from this operation, for each of the discrete design variable values. The optimal operation for a given set of design variables is found by dynamic programming in which the state variables (volumes of water in the reservoir and in the aquifer) are discretized.

For the purpose of this research, initially a set of coarse grids are used for the design variables. At each iteration, grid points are selected in the neighborhood of the optimal solution of the previous iteration. The grid spacings for the design variables are reduced at each iteration if they are not fine enough already, while keeping the grid spacing for the state variables constant. Results of five iterations with an execution time of around 71 CP minutes in CYBER 75 at Georgia Institute of Technology are given in Table 1. The net return for each design combination used in the final run (iteration 5) is given in Table 2. Note that at the point where the procedure was stopped, the optimal design variable values do not coincide with one of the end grid points. The optimal design Reservoir Capacity (Q^*) = 750,000 acre-ft, Powerplant Capacity (E^*) = 70 MW, Canal Capacity (U^*) = 1,700,000 acre-ft, and Pumping Capacity (W^*) = 375,000 acre-ft. The optimal operation corresponding to this optimal design is summarized in Fig. 4.

A limited investigation is done to check the sensitivity of the solution to the hydrological data. A new rainfall and streamflow data are generated with prolonged periods of wet and dry periods, by increasing rainfall data by 10% and streamflow data by 50% for three years, and decreasing rainfall data by 10% and streamflow data by 50% for another three years. These two periods are four years apart from each other. The rest of the data were also manipulated to keep the variances and means same with the original data. The solution obtained for this set of data is: Q^* = 750KAF; E^* = 70MW; U^* = 1925KAF; and W^* = 375KAF. The objective function value \$665.135 $\times$ 10^6, which is 6.0% less than the one obtained for the original data, which appears to be insignificant for rather significant change in the hydrological data. Further computer runs, however, are needed to make a definitive statement.

Summary and Conclusions

In this study, working mathematical model is provided to optimize the conjunctive use of surface and ground-water resources for irrigation, hydropower generation, aquifer recharge, and other mandatory requirements. The model was large, nonlinear and nonconvex, thus readily available algorithms may not solve it in a computationally satisfactory manner. Therefore, a solution strategy is developed by taking full advantage of the mathematical structure of the model and by utilizing insight about the physical system that was modeled. This approach, designing a solution procedure for the problem, rather than vice versa, is a realistic approach to engineering problems. Furthermore, the approach is mathe-

matically rigorous, as mathematical proofs of all the results based on insights about the system are obtained and given in (13).

The solution procedure was programmed in FORTRAN and was tested to obtain an optimal solution for a practical large size problem using a set of realistic data. The problem solved has 394 variables and 390 constraints, excluding the nonnegativities, the upper and lower bounds on the design variables. For a fixed design, we have 390 variables and the same number of constraints with nonconvex objective function and nonconvex feasible region. Solution time for this size problem is, on the average, 15 CP seconds on a CDC CYBER 74 computer. If only the optimal return from the operation of a fixed design is sought, then it takes, on the average, 7 CP seconds to compute the objective function value. Thus, the computer program is capable of solving large size practical problems.

The total execution time to compute the optimal design and operation was close to one hour. However, considering the dollar amount of the total investments, this much computer time (which is generally priced at approximately $200) is certainly justified. Furthermore, the computational process was an iterative one. A practitioner, experienced in the water resources, would have better insight on what constitutes "good" design values. Clearly, the better the starting point, the faster will be the convergence to the optimal solution. The reader may note that in the second iteration Q (reservoir capacity), E (powerplant capacity), and W (pumping capacity) values were relatively close to their optimal values where the U value (the canal capacity) was still far from its optimal value. A practitioner experienced in this area could have recognized this at the outset.

The computational times discussed above were achieved because of the solution strategy developed in this research. This allows us to state that recognizing the special structure of a mathematical programming problem is a very crucial step in obtaining an efficient solution algorithm for the problem.

The reader may also note that the numerical results obtained justify the study of the conjunctive use of the surface water and ground water. From the optimal operation, as summarized in Fig. 4, in wet periods the surface water usage rate increases whereas, the groundwater usage rate decreases. In dry periods, the opposite occurs. This synergism would decrease the reservoir detention requirement and also improve the overall net benefits. This conclusion confirms the intuitively expected situation.

Another point to be highlighted in this section is the refinement achieved in the solution of the dynamic programming problem. The aforementioned strategy of incorporating the transformation function in the constraint set of the stage problem to avoid need for interpolation in the output state variables, together with discretization of infeasible regions, has helped the solution procedure significantly. This scheme appears to be a novel one.

Acknowledgments

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COMPUTER MODELING OF FLOOD ALLEVIATION BENEFITS[a]

By John B. Chatterton[1] and Edmund C. Penning-Rowsell[2]

INTRODUCTION

The investigation of flood damages, and more particularly flood damages averted, already plays an essential part in the economic analysis of urban flood alleviation schemes undertaken by British regional water authorities. Indeed, such analysis is required by the Ministry of Agriculture, Fisheries and Food in support of applications for government grants in England and Wales to demonstrate that the total costs incurred by the community in alleviating flooding are matched by the benefits to be gained by the works. However, until recently the data and guidelines on potential flood damage to property at risk have been minimal.

In 1967 the British Institution of Civil Engineers recommended the systematic collection of flood damage data and its application to benefit assessments. More recently, Section 24/5 of the Water Act 1973 requires water authorities to undertake 'surveys of their areas in relation to their drainage (including flood protection) functions.' The guidance notes that clarified the Act (Ministry of Agriculture, Fisheries and Food, 1974) stressed the importance of cost-benefit procedures in future appraisals of flood alleviation and warning schemes. Similarly, the National Water Council's Standing Committee on Sewers and Water Mains, through its Working Party on the Hydraulic Design of Storm Sewers, has pointed to the need for more attention to assessing the benefits of storm drainage systems (National Water Council, 1977). The tide of opinion in this field is turning in favor of design based on cost-effectiveness rather than solely on hydraulic considerations.

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Standard Depth/Damage Data

In Britain, techniques for estimating flood magnitudes and their recurrence intervals are well advanced (Natural Environment Research Council, 1975). However, to improve the design of flood alleviation schemes it is essential to complement these techniques and data with standard, nationally applicable flood damage data so that accurate benefit analyses are possible. To this end, the Middlesex Polytechnic Flood Hazard Research Centre in collaboration with the regional water authorities and the Ministry of Agriculture, Fisheries and Food has been investigating methods of assessing and standardizing the benefits to the community of protecting urban land from flooding (Penning-Rowsell and Chatterton, 1977, 1980). In addition to providing standard damage data an interactive computer model has been developed to standardize benefit calculations and enable rapid assessments of the benefits of alternative design standards, thus facilitating sensitivity analysis. (All damage data in this paper are quoted in £ sterling and $ conversions are provided where appropriate (£1 ≑ $2.10).)

There are two basic approaches to the assessment of flood damages. First, the collection of information on damages can proceed as floods occur, and second, a theoretically-based or 'synthetic' approach can be used that involves costing likely potential damage from flood events with a range of magnitudes and durations. This second approach draws on the accumulation of diverse information and experience on flood damage from a number of past flood events. The approach was developed by White (1945) in the USA and is widely employed by the Corps of Engineers and other government agencies (Grigg and Helveg, 1975; Day and Lee, 1976). White introduced the concept of the depth/damage curve to relate potential damage in property to depth of flooding. In reality, damage is a function of not only depth, but flood duration, flood water velocity and effluent content—the hydrological variables—as well as the size and function of the property and the quality and quantity of its contents—the socio-economic variables. Nevertheless, this 'synthetic' approach was taken in the Middlesex Polytechnic research project to provide nationally applicable depth/damage information and replace *ad hoc* procedures. These, e.g., had estimated potential damage as 10% of house value or an arbitrary multiple of the property rateable value (Ministry of Agriculture, Fisheries and Food, 1974).

It is customary to distinguish between tangible and intangible flood damages based on whether monetary damages can be assigned to the consequences of flooding. The research reported herein has attempted to systematize the calculation of direct and indirect tangible damages for any sequence of flood events within a clearly defined flood prone area—the benefit area—whilst at the same time appreciating that reliance should not be placed on tangible economic benefits alone. Many other factors need careful consideration when evaluating the potential for flood protection. These include important amenity and conservation issues (Water Space Amenity Commission, 1980), personal anxiety of ill-health (Bennet, 1970) and political feeling (Penning-Rowsell and Chatterton, 1976). If the public decides that flood alleviation is needed despite an unfavorable economic analysis, then this is their choice; the ultimate responsibility for public expenditure lies with the public.

The land use of a flood prone area profoundly affects the likely flood damage characteristics. Houses are affected differently from offices and stores, which

in turn suffer different kinds of damage from those experienced in industrial premises or public parks. Nine land use types have been identified and guidelines have been produced for assessing the effect of different depths of flooding on each (Penning-Rowsell and Chatterton 1977, Appendix 1.1). The nine types are as follows: (1) Residential dwellings; (2) agricultural buildings; (3) nonbuilt-up land (nonagricultural); (4) nondomestic residential; (5) retail trading and related services; (6) professional offices; (7) public buildings and community services; (8) manufacturing and extractive industries; and (9) public utilities and transportation.

Alternative methods of obtaining flood damage information for all property types have been derived. Basically these alternatives comprise either site surveys or the use of standard 'synthetic' depth/damage information as presented by Penning-Rowsell and Chatterton 1977, Appendices 2.3–7.1) and summarized by Penning-Rowsell and Chatterton (1980). These standard data were developed by a synthesis of information on property contents and building value, flood damage susceptibility, and flood effects.

The choice of method of obtaining flood damage information depends on the size of the benefit area under investigation and the required accuracy of the assessment. A hierarchical land use classification has been developed to allow flexibility in benefit assessments. For *generalized surveys* the user can record each property as the 'average' house or store and select the appropriate aggregated depth/damage data. For *more detailed assessments* it is necessary to record, e.g., each semidetached house (one of the most common types of British house, with two properties joined by a party wall on one side only), and determine the social class of its occupants to indicate the quality and quantity of the contents. In the retail sector the user can separate a butcher's from a general store and select the disaggregated data appropriate to each of these specific properties. At the *highest level of detail* the user can conduct site surveys using standard check-lists. By applying standard susceptibility data to the inventory items thereby identified in each property, depth/damage data specific to individual buildings can be created.

Standard depth/damage data for residential property was compiled from three basic sources: (1) Typical ground floor plans (needed for the likely dimensions of each property); (2) the standard flood susceptibility data to be applied; and (3) the building fabric details and the inventory value for each property. It was impractical to mount large scale surveys to determine the ownership and quality of furniture and household durables within different types of dwellings, so secondary sources were used to derive this information. For instance, the variation in household durable ownership for the whole of Britian was derived from a market research sample of 35,000 households. Similarly, damage to retail properties was amalgamated from information from the estates departments of 41 companies representing nearly 8,000 stores, while the National Census of Distribution was a major data source on average retail stock values and store sizes. Using this information, together with the cumulative experience of members of the Chartered Institute of Loss Adjusters, of chartered surveyors and of structural engineers, enabled the collection of full and detailed information on the susceptibility of individual properties to flood damage. These data are presented by Penning-Rowsell and Chatterton (1977) as arrays relating flood damage to flood depth.

The range of standard depth/damage information thus produced is shown in Fig. 1. It should be noted that building materials in Britain are predominantly stone or brick, flood velocities are generally low and structural failure is rare. As a consequence flood damages are constant above 1.5 m. In the Bristol floods of July 1968, when 3,000 dwellings were affected to depths exceeding 2 m, no structural failure occurred.

The assessment of potential flood damage to industrial premises probably presents the most difficult problems in cost-benefit analysis of flood alleviation schemes, owing to the lack of any comprehensive past damage information and the great diversity in size, intensity, and function of industrial premises. A questionnaire has been developed, therefore, to provide information on the

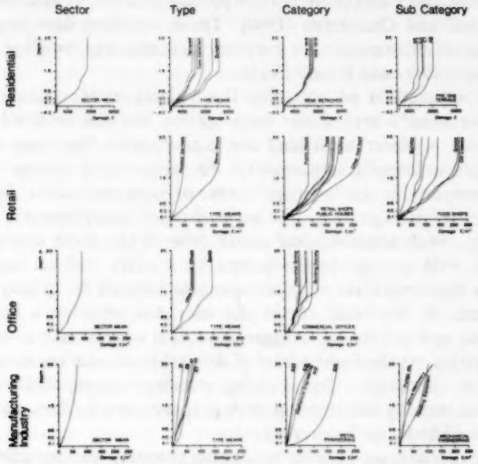

FIG. 1.—Examples of Standard Flood Depth Damage Data for Four Main Land Use Sectors (Residential, Retail, Offices, and Manufacturing Industry) Showing both Sector Average and Specific Data (£1 = $2.10)

susceptibility of premises to physical damage and the likely magnitude of disruption to production. However, if a questionnaire response is incomplete, estimates can be made either from similar types of premises successfully surveyed within the benefit area or from the 180 manufacturing and related properties, all with flood plain locations, surveyed during the course of the research (Fig. 1) (Penning-Rowsell and Chatterton, 1977, Appendix 5.1).

It is not possible to standardize the relationship between direct and indirect flood damage. Too many site-specific variables affect the disruption of roads and other communication systems to use fixed coefficients relating resulting indirect losses to physical damages, which in certain circumstances may be negligible when indirect effects are considerable. Standard data has been derived for commercial and industrial premises measuring indirect damage in terms of

loss of trading profit. This loss can be significant and, indeed, industrial flood damages can produce a multiplier effect when severe disruption of production can affect a local community's economic welfare and possibly even disturb regional and national economies. The data should, however, be used with caution since loss of profit may be recouped if production is transferred or deferred.

Computer Model

In calculating the costs and benefits of flood alleviation schemes, it is obviously desirable for water authorities and consulting engineers to follow a standard evaluation procedure, to ensure a consistent allocation of resources. The production of standard depth/damage relationships and agreed guidelines for the estimation of other benefits provides the basic potential flood damage data. When applied to the land use mix within the benefit area, and related to the

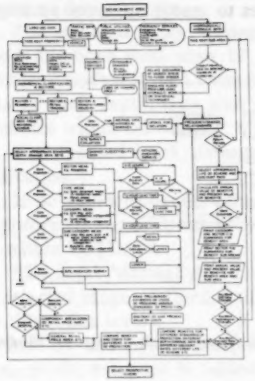

FIG. 2.—Detailed Illustration of Urban Benefit Assessment Model

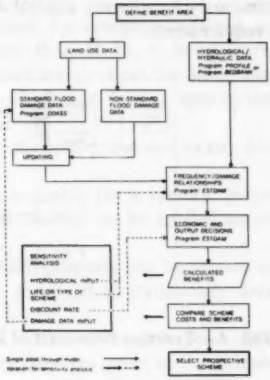

FIG. 3.—Major Components of Urban Benefit Assessment Computer Model

flood stages for different magnitude events, the annual benefits of protecting against floods of a range of recurrence intervals can be calculated. These benefits will then be comparable for all government sponsored schemes and can be compared with the costs and the intangible effects of floods and flood alleviation schemes.

A standard evaluation procedure for urban flood protection schemes is shown in Fig. 2. It should be emphasized that this evaluation procedure examines the cost-effectiveness of a range of alternative schemes rather than establishing a single benefit figure for some arbitrary design standard. When using the computer model, benefit assessments may be as detailed or generalized as required. However, the level of generality of the standard depth/damage data sets selected should correspond with the accuracy of the land use, topographic and hydrological/hydraulic data inputs. The major components of the computer model designed to operationalize the procedure in Fig. 2 are given in Fig. 3. The systems flow

chart (Fig. 4) shows the model to incorporate three separate but linked computer programs: (1) PREASS (pre-assembly); (2) DDASS (depth/damage assembly); and (3) ESTDAM (damage estimation).

These programs have been developed on a DEC-System 10 machine with a KL processor, having a capacity of 512K words. The main ESTDAM program utilizes 40K words of main memory store. A run to assess flood damage susceptibility for 250 properties, with 8 subareas, 6 return periods and full print options requires approx 10 sec of Central Processor Unit time and 1 min of elapse time.

Standard depth/damage data sets are assembled using the PREASS computer program and stored as subfiles for selection for a specific scheme by program DDASS (Fig. 4). Thereafter, it is possible to run successive benefit assessments with program ESTDAM using data for different flood durations, different flood warning lead times or upper and lower confidence limits of the standard data sets. Given the short run and elapse times the system can be operated almost continuously by altering control and data files to match different assumptions and requirements.

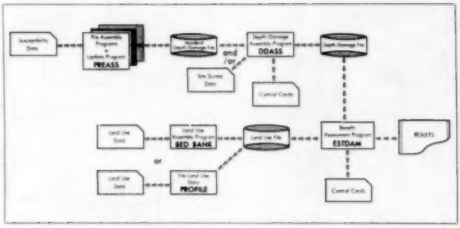

FIG. 4.—Systems Flowchart of Urban Benefit Assessment Computer Model

Certain inputs are specific to each assessment. Broadly these can be defined as: (1) Land use data for the benefit area, recording each building or property separately; (2) topographic and location data relating land use data to threshold of flooding levels and location within the benefit area. This enables each property to be related to the appropriate flood stages; and (3) flood extent, depth, and frequency data applicable to the watershed(s) concerned.

Alternative subroutines relate the flooding threshold of each property to flood stage and allocate appropriate stage/damages from information stored in the depth/damage files created in DDASS. First BEDBANK relates each property threshold to the nearest river bank/embankment height and relates this height to an assumed flood profile datum along the long profile of the river. Second, PROFILE relates the threshold of each property to single or multiple two-dimensional flood profiles either actually surveyed from historical events or statistically derived.

Within program ESTDAM the aforementioned three inputs are amalgamated with the standard damage data—suitably updated for inflation—and other nonstandard flood damage data for traffic disruption, public utilities, emergency services, and intangible effects. This amalgamation enables frequency damage

relationships and ultimately the discounted (i.e., present value of) annual benefits to be calculated as an estimate of the capital sum it would be worthwhile to invest on flood alleviation works.

Program ESTDAM is flexible so that the three basic data inputs—land use, standard depth/damage and hydrological data—can be manipulated to suit the specific requirements of the benefit assessment. The following options demonstrate this flexibility:

1. Land use of the benefit area may be coded either simply by *sector* (e.g., average residential property) or by *type* (e.g., semidetached house) or *category* (e.g., 1930s semidetached house) according to the required detail of the benefit analysis.
2. The benefit area may be divided into subareas to any specification, including the amalgamation or subdivision of predetermined subareas, so that the parts of the benefit area contributing most to the benefits may be identified for special attention at design stages.
3. The standard depth/damage data available for computer analysis is for 2 flood durations (less than 12 h and greater than 12 h), 4 flood warnings lead times (residential data only) and has standard errors about the mean damage values at each depth. The user may select whichever data is appropriate to the scheme within DDASS.
4. The standard depth/damage data can be changed or updated to any desired specification.
5. If no standard depth/damage data are available for a specific property, the appropriate sector or category average, if available, can be applied automatically within ESTDAM.
6. The depth/damage data produced for each property can be related either to a selected range of flood profiles actually surveyed or statistically derived, or to river bank levels.
7. Indirect and other damages (e.g., road traffic disruption costs, loss of trading profit, etc.) can be added to the direct damages prior to the calculation of average annual benefit and subsequent discounting.
8. Damages are converted to discounted benefits for any standard of protection scheme, for any length of scheme life, for any discount rate.
9. An overall up-dating option is included to allow for inflation and other price changes.
10. Suppression of any of the printout is possible (subarea results, area frequency/damage totals, discounted benefit tables, etc.).

This model has been extensively tested by the Severn-Trent Water Authority (e.g., Chatterton, Pirt and Wood, 1979) and has also been used by the Southern and Wessex Water Authorities and by the Greater London Council.

Sensitivity Analysis

The accuracy of the benefit estimation by this model is dependent on many factors. These include the accuracy of the land use surveys of the benefit area, the appropriateness of the standard depth/damage data and the reliability of the meteorological and hydrological data used to determine the return periods

of the various flood extents and depths. However, one of the main advantages of computer analysis is the opportunity it creates for rapid sensitivity analysis by rerunning the model with alternative assumptions and data. This allows the present value of benefits to be compared from each successive analysis and permits an informed judgement to be made on the most appropriate benefit calculation to compare with scheme costs.

Some of the variables that have a significant influence on the present value of the following benefits are given: (1) Flood extent simulations and their assigned return periods; (2) level of aggregation of land use data; (3) selection of depth/damage information; and (4) selection of discount rate and scheme life.

Thus, an incorrect simulation of flood extent could lead to incorrect inclusion or exclusion of properties within the benefit area. An underestimation of the assigned return period for a particular flood stage may increase the calculated benefits substantially. Adjustment of data items or control statements in ESTDAM allows the effect of each of these variables on calculations benefits to be explored. Equally important are the decisions made when selecting the depth/damage data subfiles within DDASS. Using depth/damage data long flood duration will increase calculated benefits while selection of data for long warning lead times will reduce calculated benefits. With interactive use both data and program controls can be modified and the model re-run within seconds.

Application of Model in Ashton Vale, Bristol (United Kingdom)

The effect of selecting different variables and data sets in this manner on the stability of calculated benefits is shown in a benefit study within Ashton Vale, Bristol (Fig. 5). Following repeated flooding in Ashton Vale—most severe in 1968—a series of culverts, tunnels, and trapezoidal channels had been constructed by Wessex Water Authority to contain storm discharges with a recurrence interval of one in 100 yr. The previously mentioned benefit model was used to provide an estimate of part of the benefits to be derived from such a scheme. In particular, an assessment was made of the benefits of protecting against floods up to the magnitude of the 1968 event, which was estimated by the Water Authority to have a 64-yr recurrence interval (Penning-Rowsell and Chatterton, 1977; Chapter 8).

Techniques and Assumptions.—In complex urban areas where many streams are culverted and sewerage outfalls are an integral part of the drainage system, flood discharges from events of similar frequencies and magnitudes can have quite different depth/extent relationships. It is difficult to predict the extent and depth of a flood of any specific frequency, yet this is crucial to accurate benefit assessment. A flood of the 1968 magnitude is unlikely to cover exactly the same area again in Ashton Vale even assuming the tidal situation in the river Avon is repeated. However, reliable 1968 flood depths in 262 dwellings are available and to overcome some of the problems of local hydraulic anomalies a generalized flood profile was computed using trend surface analysis (O'Leary, Lippert, and Spitz, 1966) to represent a likely future flood comparable to the 1968 event. This simulated profile was lowered differentially within subarea—reaches exhibiting similar hydraulic characteristics—to match flood extents for known lesser floods.

The ESTDAM benefit assessment program was used to extract and sum the

appropriate depth/damage data for each property within each land use sector to produce expected damages for each of three flood events which were initially designated as the 26 yr, 40 yr, and 64 yr floods. After addition of nonstandard data (indirect damages) the stage/damage relationships were converted to discounted annual benefits using the then standard British 10% treasury discount rate and a scheme life of 25 yr. Mean values of depth/damage data assuming no flood warning and less than 12 h flooding had been used as input data to DDASS.

Using the benefit assumptions made above as a 'standard' or reference datum, the data sets and control parameters have been altered to illustrate the sensitivity of the calculated benefits to both economic and hydrological decisions. Table 1 summarizes the results of repeated benefit calculations using alternative data

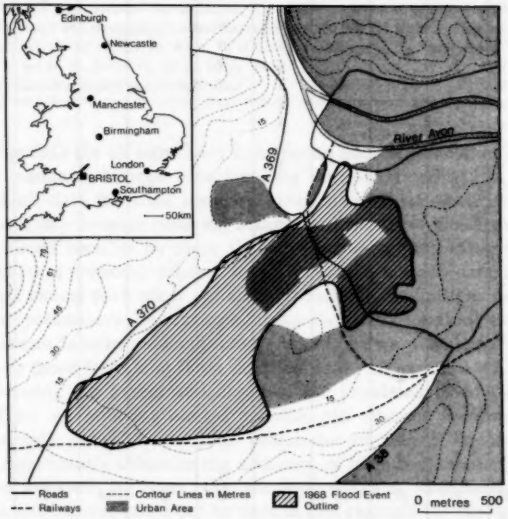

FIG. 5.—Extent of 1968 Flood Event in Ashton Vale, Bristol, United Kingdom

sets with different economic and hydrological parameters. Table 2 ranks the specific parameters which contribute to increasing or decreasing the benefits.

Of considerable significance here is the choice of discount rate, which must contain an element of opinion or value judgement. A change from a 10% rate to a 5% rate, as implemented recently in Great Britain for the appraisal of flood alleviation and land drainage schemes, can almost double the present value of benefits. However, a decision to change the test discount rate is political and outside the scope of decision-making by the engineer or economist involved in benefit evaluation.

Thus, apart from the variations in benefits caused by the discount rate, the accuracy of hydrological data, the quality of land use data and the type of

TABLE 1.—Present Value of Benefits using Alternative Sets of Parameters

Parameter Changes											
Present value of benefits, in British pounds (1)	Percent of 'Standard' parameter benefits[a] (2)	Flood duration[b] (3)	Flood warning[c] (4)	Depth/damage data[d] (5)	Land use data[e] (6)	Evacuation of vehicles[f] (7)	Flood extent[g] (8)	Other benefits[h] (9)	Return periods[i] (10)	Life of scheme, in years (11)	Discount rate, as a percentage (12)
669,000	216.9								2		
456,600	148.1	2			*S*	*N*				50	9
446,300	144.7			*U*	*S*	*N*				50	9
413,000	133.9	2			*S*	*N*				50	
403,600	130.9			*U*	*S*	*N*				50	
375,800	121.8	2			*S*						
367,400	119.1			*U*	*S*						
356,900	115.7	2								50	
355,000	115.1				*S*						
353,500	114.6	2									9
353,400	114.3							*E*			
347,200	112.6			*U*						50	
344,000	111.5			*U*							9
342,700	111.1			*L*	*S*						
336,900	109.2									50	
335,400	108.8	2	4		*S*						
333,700	108.2										9
326,700	105.9	2									
325,600	105.6			*L*						50	
322,800	104.7		4		*S*						
322,500	104.7			*L*							9
317,900	103.1			*U*							
315,600	102.3	2	4							50	
312,700	101.4	2	4								9
310,600	100.7					*N*					
308,400	100.0	1	*N*	*M*	*F*	*Y*	*S*	68	1	25	10
303,500	98.4		4							50	
300,700	97.5		4								9
298,100	96.6			*L*							
289,000	93.7	2	4								
282,300	91.5	2									12
280,000	90.8		4			*N*					
277,900	90.1		4								
274,700	89.1			*U*							12
266,500	86.4										12
257,500	83.5			*L*							12
249,700	81.0	2	4								12
240,100	77.9		4								12
161,200	52.3								3		
105,100	34.1						*L*				
92,800	30.1		4				*L*				

[a] 'Standard' parameters = benefits as calculated assuming protection against all events up to and including 1968 flood in Ashton Vale. Parameters assume no warning and rapid dispersal of flood waters after tide levels in Avon are reduced.

TABLE 1.—*Continued*

[b]Flood duration 1 = less than 12 h, 2 = greater than 12 h.

[c]Flood warning = *N*—no warning, 4—4 h effective lead time.

[d]Depth/damage data = *U*—upper confidence limit (95% level) of selected data set, *M*—mean of selected data set, *L*—lower confidence limit (95% level) of selected data set.

[e]Land use data = *S*—sector averages used as calculated from depth/damage data files, *F*—full land use code given and appropriate depth/damage data selected.

[f]Evacuation of vehicles = *N*—vehicles assumed not moved before flooding, *Y*—vehicles assumed moved (each residential property is assigned 0.7 × vehicle in accordance with national car ownership statistics).

[g]Flood extent = *S*—'standard' extent as estimated from trend surface analysis and adjusted to simulate actual flood extents corresponding to return period 1, *L*—profile lowered by 0.3 m in all subareas.

[h]Other benefits = 68—indirect etc., damages added to highest magnitude flood as estimated from 1968 event (£288,000) and updated for inflation, *E*—cruder estimation adding smaller amounts to lesser magnitude floods and increasing indirect damages to £500,000 for highest magnitude flood.

[i]Return periods = 1—12 yr, 25 yr, 40 yr, and 64 yr as estimated for four flood extents used, 2—5 yr, 15 yr, 30 yr, 64 yr, 3—20 yr, 30 yr, 50 yr, 64 yr.

Note: Blanks indicate standard parameters used.

depth/damage data are all significant in affecting benefit calculation. In Ashton Vale, using standard damage data for long duration flooding, or the upper confidence limit data from the standard error calculations—both assuming no provision for likely damage reducing actions as a result of flood warning—increases calculated benefits by more than 30% when applied to sector average land use data for a proposed scheme with a 50-yr life. Certain parameters when taken together cancel each other out and approximate the benefits derived from the 'standard' parameters. For example, using short duration flood damage data, which assumes no damage-reducing actions from flood warnings, produces results comparable to those from long flood duration data that assumes effective flood warnings. Altering the extent and depth of the simulated flood profile has a profound effect on reducing annual benefits and stresses the importance of accurate hydraulic data. The accurate calculation of indirect benefits for each flood stage significantly enhances the accuracy of benefit assessments.

It must be stressed, however, that the long duration, confidence limit and warning data alternatives could not be used in this example for the flood damage estimates for the industrial and much of the retail-related service property (warehouses, etc.). Damage data specific to these properties were collected during questionnaire-based site surveys. The true effect of varying each of the parameters discussed above is therefore masked, since a substantial proportion of the benefits is derived from the industrial and related sectors (67% in the 'standard' parameter calculation). Sensitivity analysis, therefore, is properly revealed here only for the residential sector. Table 2 shows that for this sector alone event damage for the 1968-type flood is increased by 27% using long duration flood damage data, 12% using upper confidence limit data, and decreased to 49% of 'standard' parameter damage using data assuming an effective 4-h warning lead time.

Effect of Land Use Data Precision on Calculated Benefits.—Using generalized land use data produces a benefit assessment recording each residential property as the overall 'average' dwelling and each retail property as the overall 'average'

store. There is, e.g., no differentiation between bungalows (single storey dwellings) and semidetatched houses or between butchers' and general stores. With this approach average depth/damage data for all industrial properties in Ashton Vale was applied to each factory or warehouse at risk.

Table 1 shows that this approach increases likely future benefits by about 15%. This is mainly for two reasons. First, the residential sector average over-estimates likely damages in this area where poorer quality 1930s semidetached housing largely predominates; yet, the overall sector average is compiled from a complete cross section of British properties. The difference between the generalized and the detailed benefits for the 64-yr flood in the residential sector is some £65,000 ($136,500) or an increase of 7.5%. Thus, in a poor area this sector average approach will raise calculated benefits and in a richer area it will depress them. Second, and more important, with the generalized

TABLE 2.—Effect of Specific Parameters on Changing Present Value of Benefits

	Parameter change (1)	Percent of 'standard' present value of benefits (2)		Percent of 'standard' 64-yr event damages (residential sector only) (3)
1	Sector average land use data	115.1		107.5
2	(Increase indirect benefits)	(114.3)		
3	50-yr scheme life	109.2	In-	
4	9% discount rate	108.2	crease	
5	Long flood duration	105.9	annual	127.2
6	Upper confidence limit data	103.1	benefits	112.0
7	Evacuation of vehicles	100.7		103.9
8	4-h flood warning lead time	98.4		49.0
9	Lower confidence limit data	96.6	De-	88.0
10	12% discount rate	86.4	crease	
11	Reduction of flood extent/depth	34.1	annual benefits	

Note: Parameters 1–8 represent increases in annual benefits, 9–11, decreases in annual benefits.

approach there is an increase of industrial benefits, with a difference of £430,000 ($903,000), or an increase of 66%. This discrepancy arises from the overweighted overall sector average per square meter of floor space being biased towards the small, intensively used workshops with high damage potential. When this average is applied to the larger factories, where space is used less intensively, their damage potential is grossly exaggerated. This points once again to the need for individual site surveys in industrial premises.

Effect of Return Period Estimates on Discounted Benefit Calculations.—The figures in Table 1 include calculations only up to the 64-yr level, which is the return period of the 1968 event calculated by Wessex Water Authority (no information was available for the extent of larger floods and simulation of such flood extents is difficult in this complex urban area). Therefore the annual benefits as calculated may be lower than in reality for the scheme as designed

since the contributions of the larger floods to the benefit total are not included.

If the return periods of the lesser magnitude floods are altered from the best estimates the effect on the calculated annual benefits and subsequent discounted benefit totals is very profound. For example, if the largest flood is maintained as the 64-yr event but the return periods of the lesser events altered from 12 yr, 25 yr, and 40 yr–5 yr, 15 yr, and 30 yr the discounted benefits increase to £669,000 ($1,404,900) or over double the figure from the best estimate of return periods. Again, increasing the return periods of the same lesser floods to 20 yr, 35 yr, and 50 yr reduces the discounted benefits to only £161,200 ($338,500). These alterations of the lower return periods were undertaken to demonstrate their particular effect on benefit calculations. Clearly, the calculated economic viability of flood alleviation scheme may depend critically on the probabilities assigned to quite minor events, which generally receive somewhat scant attention from the design engineer. Perhaps fortunately the return period and discharge estimates for these smaller events, given limited historical rainfall or flow records, are likely to be more accurate than those of the larger events. However, information on flow extents at these low return periods is often sparse, given the natural inclination of engineers to concentrate on recording the discharge and extent of the largest known flood.

Changing in isolation the return period estimate of the largest event under consideration should be undertaken with some caution. Comparisons between benefit tables can be distorted when using different flood return period series. An over-estimate of the return period of the largest event will enhance the present value of calculated annual benefits. However, in many locations the accuracy of annual benefit calculations depends rather more on the accuracy of low magnitude, high frequency flooding than vice versa. Here the precise estimate of the frequency of the largest flood is not as significant as the estimate of the lesser events also to be protected against by alleviation measures.

Conclusion

When designing storm drainage or flood protection schemes it is preferable to optimize economic efficiency through maximizing the difference between benefits and costs by evaluating several alternative design standards. This optimization is very tedious when calculations are undertaken manually and many authors recognize that computer calculations greatly facilitate the evaluation and design process (United States Army Corps of Engineers, 1977, 1979, 1980; Davis and Webb, 1978; Smith and Greenaway, 1980).

The aforementioned computer model facilitates the design process by enabling the decisionmaker to select that calculation of discounted benefits most appropriate to the design flood conditions and the proposed scheme. An important feature of the model is its automated incorporation of large amounts of nationally applicable flood damage data and the facility to add site-specific damage data obtained from questionnaire or valuation surveys. In addition a number of alternative methods is available for relating properties at risk to flood profile data, as are optional subroutines producing a full discounted cash flow analysis of both scheme costs and benefits. The model is now interactive so that the data sets and the control options fixing the economic and hydrologic assumptions for each assessment may be readily changed for sensitivity analysis.

Using the full facilities of the model assists the design process in a number of ways. First, many different design standards can be evaluated and a choice of standard made on the basis of this evaluation rather than using some predetermined flood return period. Second, errors in economic evaluation are minimized through automated calculations. Third, the effect of the many assumptions on calculated benefits can be explored and those most critical to the final results can be subject to the most scrutiny. In these and other ways the evaluation of both structural and nonstructural flood alleviation schemes can be made faster, more comprehensive and more accurate.

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DECISIONS UNDER VIOLATION OF REGRESSION NORMALITY

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INTRODUCTION

Many hydrologic decisions depend on the prediction of future values of variables that influence the outcome of the decision. These future values are predicted frequently on the basis of multiple linear regression analysis. For example, culverts are designed as a function of peak T-year flow (35), which may be estimated by means of a regression on watershed and climatological characteristics (23,30). Two other examples are the determination of flood plains (5,14) and the design of a reservoir subject to sedimentation (9,13) for which sediment yield is often estimated by means of multiple linear regression (4,10,11,20,33).

In all the references cited thus far, classical regression analysis was used. Modern methods of data network design can involve a combination of regression analysis and simulation (24,25). However, few papers propose to imbed the estimation error associated with regression or the variance of the predicted dependent variable in a loss function, although the advantages of using a loss function for decision-making have been pointed out (1). Two notable exceptions include: (1) The prediction of phosphorus loading into the Great Lakes by means of regression (26) in which work it is suggested to use the regression error in an economic decision model; and (2) the use of hypothesized loss functions to assess the value of information to flood frequency analysis (28).

Classical inference in multiple regression analysis is based on the assumption that the error distribution is normal. This assumption is violated for many types of data encountered in engineering and hydrology. In the present paper, it is shown that many hydrologic decisions, which include, at least implicitly, the variance of the forecast, are very sensitive to nonnormality of errors (measured by skew).

The purposes of this study are, first, to investigate the small sample behavior

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of the estimated regression coefficients, estimated variances, and predicted values of the dependent variable when the errors are not normally distributed and, second, to show how losses due to wrongly assuming normality can be calculated.

It should be emphasized that the statistical properties of extreme hydrologic variables, which have been studied extensively in the literature (3,6,16,31,32), are investigated herein from the explicit viewpoint of regression analysis and then decision-making. In other words, it is assumed that the state variable of interest—the flood stage—is estimated through regression analysis rather than direct observation.

For the first purpose, a simulation methodology is used to vary the mean and variance of a predicted variable. For the second purpose, two hydrologic decision-making situations are considered. One with a symmetric objective function and the other with a nonsymmetric objective function. The simulation approach is used to estimate means and variances of these loss functions.

The following section introduces the methodology in terms of the theoretical model and assumptions, the decision examples, and the procedures. The subsequent section presents and analyzes the results. Finally, a discussion of methodological problems is given, and conclusions are drawn.

Problem

Theoretical Elements.—The classical linear regression model (14) is given by $\mathbf{y} = \mathbf{x}\,\boldsymbol{\beta} + \boldsymbol{\epsilon}$ in which $\mathbf{y}$ is an $\mathbf{n} \times \mathbf{1}$ vector of the n observations of the dependent variable, $\mathbf{x}$ is an $\mathbf{n} \times \mathbf{p}$ matrix of the n observations of the p independent variables, $\boldsymbol{\beta}$ is a $\mathbf{p} \times \mathbf{1}$ vector of regression parameters and $\boldsymbol{\epsilon}$ is an $\mathbf{n} \times \mathbf{1}$ vector of errors. For this model, $\mathbf{y}$ is an observable random variable and the errors are assumed to have mean 0 and covariance matrix $\sigma^2\mathbf{I}$. The model is also assumed to be full rank, i.e., $\mathbf{x}'\mathbf{x}$ is nonsingular. Under these assumptions, the least squares estimates of the parameters, given by $\hat{\boldsymbol{\beta}} = (\mathbf{x}'\mathbf{x})^{-1}\mathbf{x}'\mathbf{y}$, are the minimum variance linear unbiased estimates. If the errors are normally distributed, the parameter estimates $\hat{\boldsymbol{\beta}}$ are multinormally distributed with mean $\boldsymbol{\beta}$ and covariance matrix $\sigma^2(\mathbf{x}'\mathbf{x})^{-1}$, and $(n - p)\hat{\sigma}^2/\sigma^2$ is distributed as chi square with $n - p$ degrees of freedom in which $\sigma^2 = \mathbf{y}'(\mathbf{I} - \mathbf{x}(\mathbf{x}'\mathbf{x})^{-1}\mathbf{x}')\mathbf{y}/(n - p)$.

Thus, when the errors are normally distributed, confidence intervals and statistical tests of hypotheses can be constructed. For large samples, the Central Limit Theorem assures that the tests and confidence intervals are approximately correct. For small samples, the Central Limit Theorem does not apply, and the question of sensitivity of the classical results to nonnormality must be considered in basing decisions on those results. This study concerns the small sample behavior of the estimated regression coefficients, estimated variance, and predicted values of the dependent variable when the errors are not normally distributed. The cost of incorrectly assuming normality is evaluated for two hydrologic decision situations in terms of the decisions to be made and their associated loss functions.

Decision Example.—The analyses are based on the results of Monte Carlo simulations using the data discussed by Reich (27). These data were obtained for the purpose of estimating parameters of hydrographs in very small watersheds using linear regression analysis. In Reich's analysis, runoff, rainfall, and infiltra-

tion are assumed to be related by the equation

$$y_i = \beta_0 + \beta_1 x_{1i} + \beta_2 x_{2i} + \epsilon_i \quad \ldots \ldots \ldots

The $(1 - \alpha)$ upper confidence limit for y is given by

$$\hat{u} = \mathbf{x}_0\hat{\boldsymbol{\beta}} + t_{n-p,\alpha}\{[1 + \mathbf{x}_0'(\mathbf{x}'\mathbf{x})^{-1}\mathbf{x}_0]\,\sigma^2\}^{1/2} = \mathbf{x}_0\hat{\boldsymbol{\beta}} + K_\alpha\hat{\sigma} \quad \ldots\ldots\ldots (5)$$

in which $K_\alpha = t_{n-p,\alpha}[1 + \mathbf{x}_0'(x'x)^{-1}\mathbf{x}_0]^{1/2}$ and t is Student's t. $\ldots\ldots$ (6)

The *pdf* of $\hat{u}$ is computed as the convolution of the *pdf*'s of $\mathbf{x}_0\hat{\boldsymbol{\beta}}$ and $K_\alpha\sigma^2$. The estimates $\hat{d} = \mathbf{x}_0\hat{\boldsymbol{\beta}}$ and $\hat{u} = \mathbf{x}_0\hat{\boldsymbol{\beta}} + K_\alpha\hat{\sigma}$ are used in the example involving the loss incurred for violating the normality assumption.

The simulation experiments were performed as follows. A set of n sample

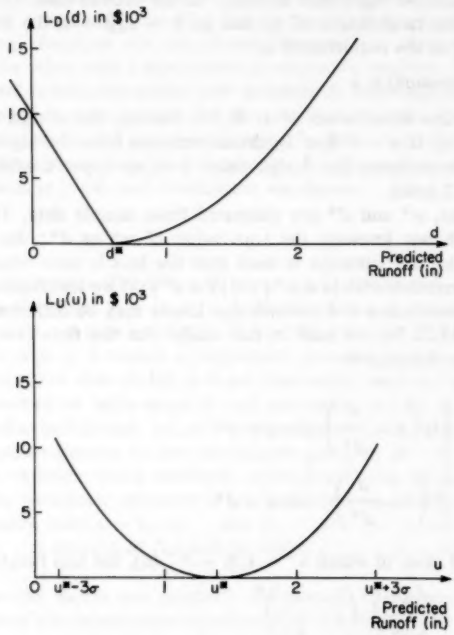

FIG. 1.—Loss Functions Used in the Simulation Studies

points was taken from the data of Reich (27) and the regression coefficients $\hat{\beta}_0^*$, $\hat{\beta}_1^*$, and $\hat{\beta}_2^*$ were computed. Then 1,000 samples of size n were generated for each case, using the model

$$y_i = \hat{\beta}_0^* + \hat{\beta}_1^* x_{1i} + \hat{\beta}_2^* x_{2i} + \epsilon_i \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which ϵ_i had one of four distributions: normal, lognormal, gamma, or log-Pearson III. A total of 37 cases were simulated for $n = 21$ and for $n = 47$. These simulations involved lognormal, Pearson III, and log-Pearson III distributions with $E(\epsilon) = 0$ and $\sigma(\epsilon) = 0.08$, 0.257, 0.80 and 2.00. The logPearson III case for $\sigma = 2$ was omitted, since its skew λ is undefined (3). Additional

runs were performed using three replications each of lognormal distributions with $\lambda = 3, 5, 7, 8, 9, 10, 11, 12$. These distributions are summarized in terms of their type and skew in Table 1. Note that the distributions given in Table 1 are the distributions from which observations are drawn. The skew possible for any size sample is subject to the algebraic bound: $(n - 2)/(n - 1)^{1/2}$ (16). The normal and lognormal variates were generated using the computer library routine GETRAN, available at the University of Arizona Computing Center. The Pearson III and log-Pearson III variates were generated using Johnk's algorithm as described in Berman (2).

One indication of nonnormality of a probability distribution is its skew, λ, defined as $\lambda = \mu_3/\sigma^3$ in which μ_3 is the third central moment of the distribution and σ is its standard deviation. Skew measures the degree of asymmetry of a distribution and is used in this study as an indication of nonnormality. Previous studies (34) indicate that regression estimates may be sensitive to skew. Only positively skewed distributions are considered, since most nonnormal distributions encountered in hydrology and engineering involve positively skewed observations

TABLE 1.—Simulated Distributions

Type of distribution (1)	Skewness (2)
Normal	$\lambda = 0$ (4 cases)
Lognormal	$\lambda = 0.24, 0.79, 2.91, 3$ (3 cases), 5 (3 cases), 7 (2 cases), 8 (3 cases), 9 (3 cases), 10 (3 cases), 11 (3 cases), 12 (3 cases), 14 (1 case)
Pearson	$\lambda = 0.16, 0.51, 1.60, 4.00$
Log-Pearson	$\lambda = 1.97, 5.30$

(12). Robustness of the estimates of $\hat{\beta}$ and $\hat{\sigma}^2$ to nonnormality, indicated by skew, is studied by comparing the empirical results obtained using nonnormal distributions with the results expected under the assumption of normality.

For each sample, $\hat{\beta}$, $\hat{\sigma}^2$, $\hat{d}(x_0)$, $\hat{u}(x_0)$, $L_d[\hat{d}(x_0)]$ and $L_u[\hat{u}(x_0)]$ were obtained. The empirical means and standard deviations were computed for all the variables, and the empirical distributions were obtained for all the variables except the losses.

Analyses of Results

Generalities.—The results of the analyses are discussed in three subsections. The first subsection concerns estimated means and variances of the estimates obtained from or based on the regression equations: $\hat{\beta}_1$, $\hat{\beta}_2$, $\hat{\beta}_3$, $\hat{\sigma}^2$, $\hat{d}(x_0)$ and $\hat{u}(x_0)$. The second subsection concerns means and variances of the two loss functions $L_d(d)$ and $L_u(u)$.

The estimated means and variances of the variables are considered in relation to their theoretical values assuming the errors are normally distributed. Graphs

are drawn with λ on the abscissa and $E_A[\cdot]/E_T[\cdot]$ or $\sigma_A[\cdot]/\sigma_T[\cdot]$ on the ordinate (Figs. 2–5) were $E_A[\cdot]$ denotes the actual average value of a variable obtained from 1,000 simulations of a particular distribution and $E_T[\cdot]$ denotes its theoretical value assuming normally distributed errors; $\sigma_A[\cdot]$ and $\sigma_T[\cdot]$ are similarly defined for the corresponding standard deviations. A relative increase (or decrease) in a variable, as λ increases, thus, refers to an increase (or decrease) in the ratio of the actual value to the corresponding theoretical value.

Robustness of Regression Estimates.—The properties of the $\hat{\beta}_i$'s and $\hat{d}(\mathbf{x}_0)$ are

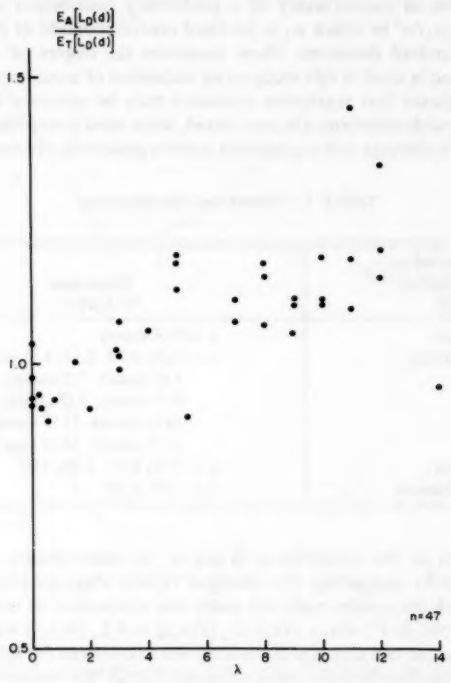

FIG. 2.—Relative Expected Values of $L_D(d)$ for Different Values of λ

very robust to the types of nonnormality studied, which tends to justify the wide use of regression analysis in hydrology and water resources as shown in Haan (12). In most cases, the empirical values are within ±10% of their theoretical values, with better agreement for the larger sample size ($n = 47$). The empirical distributions of the $\hat{\beta}_i$'s were compared with the corresponding theoretical distributions assuming normality, using the Kolmogorov-Smirnov (K-S) goodness-of-fit statistic. There is a consistent but not significant increase in the K-S statistic as skew increases. The effect of skew on the 97.5 percentile

of $\hat{d}(x_0)$, when compared with the corresponding theoretical value, was also considered. As skew increases, the theoretical upper confidence limit is overestimated by the actual confidence limit. This bias is smaller for the larger sample size.

The relative expected value of $\hat{\sigma}^2$ is not appreciably affected by skew of the error distribution; however, its relative standard deviation increases rapidly as skew increases. The overall behavior of $\hat{\sigma}^2$ is indicated by the K-S statistic,

FIG. 3.—Relative Standard Deviations of $L_D(d)$ for Different Values of λ

which increases rapidly with skew of the error distribution. Consistent with the increase in its variance, the distribution of $\hat{\sigma}^2$ is flatter for larger λ and its 97.5 percentile may be as large as three times the theoretical value for a skew of 14.

The decision variable u, based on both the mean and the variance (Eq. 4), is more sensitive to nonnormality than the decision variable d, based only on the mean (Eq. 3). Being closely related to σ^2, the relative variance of u also increases considerably with skew of the error distribution. Note that hydrologic

design is often based on variables of type u, involving not only the mean of an extreme value, such as a flood, but also its variance. The consequences of this practice should be examined in view of the results of the present analysis.

Robustness of Loss Functions.—In the first case, when the decision is based on the mean runoff, the effect of skew on the loss function (Eq. 3) is not strong. There is a slight tendency for the relative expected losses to increase in the case of nonnormality, as shown in Fig. 2. The expected losses may

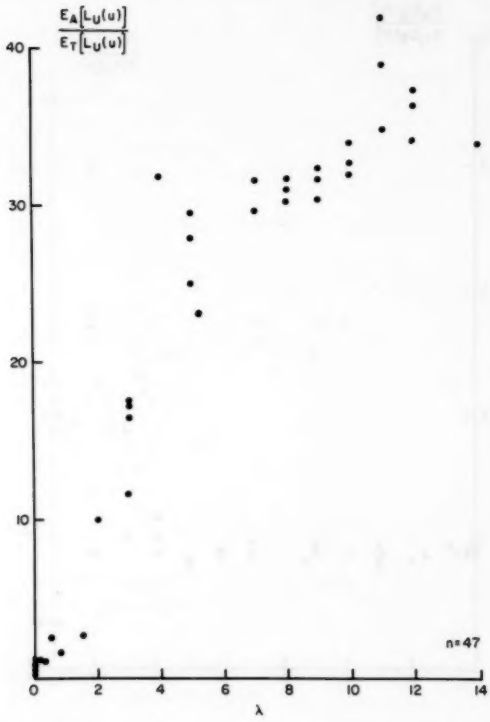

FIG. 4.—Relative Expected Values of $L_D(d)$ for Different Values of λ

be up to 10%–20% larger than in the corresponding theoretical cases for large skew ($\lambda > 10$). The standard deviation of the losses is relatively insensitive to moderate values for skew ($\lambda < 10$); on the other hand, for large values of λ combined with small sample sizes, the standard deviation of the losses may be a few times larger than in the normal case, as shown in Fig. 2. Thus, hydrologic decisions based on loss functions, which depend only on mean runoff are fairly robust to nonnormality, but the uncertainty in the loss or expected

(economic) opportunity loss (8,18,29) becomes quite large for strongly skewed distributions.

In the second case, when the decision takes into account variability as well as mean runoff (Eq. 4), the cost of incorrectly assuming normality may be high. This occurs because the estimated variance $\tilde{\sigma}^2$, which enters into the decision increases substantially as a function of skew of the error distribution. The expected value of the losses is many times that for the normal case, even for moderate values of λ, say, $\lambda < 6$, as shown in Fig. 4. The variance of the losses is also extremely sensitive to skew, as shown in Fig. 5. The implications

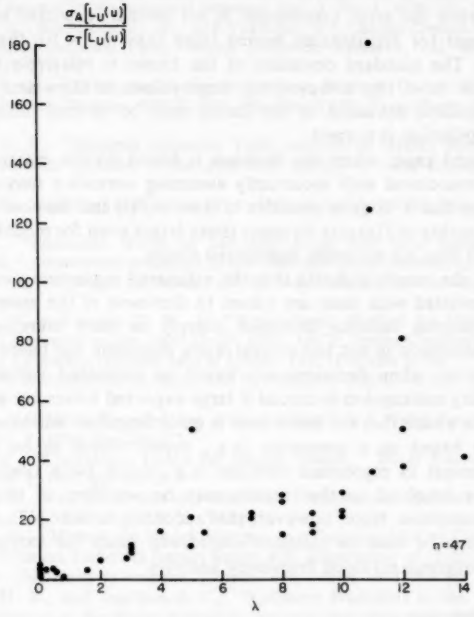

FIG. 5.—Relative Standard Deviations of $L_D(d)$ for Different Values of λ

for hydrologic design are clear: the expected economic opportunity loss is high because of overdesign or underdesign whenever the decision is based on a percentile such as 95% runoff (or 20-year flood) for culvert design or the confidence level of a regression for flood plain determination, since the determination of either a percentile or a confidence band involve both mean and variance. These types of decisions occur quite frequently in hydrology.

Conclusions

Regression is an irreplaceable tool, but it should be used with caution whenever the assumptions of a normal model are not satisfied (33). The choice of a

regression approach rather than use of another method should be viewed in the broader context of model choice (4,19). Even the choice of specification of a model should be made with due attention to various pitfalls (21,34). Should a regression model be chosen, it is recommended that the error be introduced into economic loss functions as recommended in Reckhow (26). Otherwise, the predicted variable is used in a nonoptimal fashion leading to various opportunity losses as described in Davis et al. (7).

From a decision viewpoint, the results of the study lead to the following conclusions. In the first case, when the decision is based on average runoff, the effect of skew is not serious. There is some tendency for expected losses to increase when the error distribution is not normal; expected losses may be 10%–20% larger for distributions having large skew ($\lambda > 10$) than for normal distributions. The standard deviation of the losses is relatively insensitive to moderate skew ($\lambda < 10$); however, for large values of skew and small sample sizes, the standard deviation of the losses may be several times larger than when the distribution is normal.

In the second case, when the decision is based on the estimated variance $\hat{\sigma}^2$, the loss associated with incorrectly assuming normality may be high. The results indicate that $\hat{\sigma}^2$ is quite sensitive to skew of $f(\epsilon)$ and the cost of incorrectly assuming normality of $f(\epsilon)$ may be many times larger even for moderately skewed errors ($\lambda < 6$) than for normally distributed errors.

In general, the results indicate that the estimated regression coefficients and variables associated with them are robust to skewness of the error distribution; however, estimated variance increases sharply as skew increases. Thus the normality assumption is not too critical when decisions are based on expected values; however, when decisions are based on estimated variances, validity of the normality assumption is crucial if large expected losses are to be avoided. One should be aware that the latter case is quite frequent: whenever hydrologic decisions are based on a percentile (e.g., T-year flood) or on a confidence interval statement in regression analysis (e.g., flood plain zoning), standard deviations are involved so that results may be sensitive to violation of the normality assumption. Note, however, that according to Kite (17), the normality assumption may be used to estimate confidence limits for extreme events in ordinary (nonregression) flood frequency analysis.

Acknowledgment

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Appendix II.—Notation

The following symbols are used in this paper:

$\hat{d}$ = estimate of d^*;
d^* = true value of design based on mean rainfall;
$\hat{d}(\mathbf{x}_0)$ = sample estimate of d for observation vector $\underline{x}_0$;
$E_A[\cdot]$ = actual (sample) mean of a variable;
$E_T[\cdot]$ = theoretical mean of a variable assuming normality;
F_ϵ = cumulative distribution of ϵ;
$\mathbf{I}$ = identity matrix;
L_d = loss function associated with design d^*;
L_u = loss function associated with design u^*;
$L_d[\hat{d}(\mathbf{x}_0)]$ = sample estimate of L_d for observation vector $\mathbf{x}_0$;
$L_u[\hat{u}(\mathbf{x}_0)]$ = sample estimate of L_u for observation vector $\mathbf{x}_0$;
n = number of observations;
$\mathbf{0}$ = null matrix;
p = number of independent variables;
$t_{n-p,\alpha}$ = value of student's t for n-p degrees of freedom at level α;
$\hat{u}$ = estimate of u^*;
u^* = true value of design based on distribution of rainfall;
$\hat{u}(\mathbf{x}_0)$ = sample estimate of u for observation vector x_0;
$\mathbf{x}$ = $\mathbf{n} \times \mathbf{p}$ matrix of n observations of p independent variables;
x_1 = storm total rainfall;
x_2 = infiltration capacity;

$\mathbf{x}'\mathbf{x}$ = covariance matrix of observations;
$\mathbf{y}$ = $\mathbf{n} \times \mathbf{1}$ vector of n observations of dependent variable;
y = runoff;
$\boldsymbol{\beta}$ = $\mathbf{p} \times \mathbf{1}$ vector of regression parameters;
$\hat{\boldsymbol{\beta}}$ = least squares estimate of $\boldsymbol{\beta}$;
$\mathbf{K}_\alpha$ = shorthand for expression $t_{n-p,\alpha}\,[1 + \mathbf{x}_0'(\mathbf{x}'\mathbf{x})^{-1}\mathbf{x}_0]^{1/2}$;
λ = skew;
μ_3 = third central moment of distribution;
$\boldsymbol{\epsilon}$ = $\mathbf{n} \times \mathbf{1}$ vector of errors;
$\boldsymbol{\Sigma}$ = covariance matrix of errors;
$\sigma_A[\cdot]$ = actual (sample) standard deviation of a variable;
$\sigma_T[\cdot]$ = theoretical standard deviation of a variable assuming normality;
σ^2 = variance of errors; and
$\hat{\sigma}^2$ = least squares estimate of σ^2.

MANAGEMENT OF INTERSTATE AQUIFER SYSTEMS[a]

By Harvey O. Banks,[1] Hon. M. ASCE

INTRODUCTION

There are numerous situations in the United States where aquifers are more or less continuous and the ground water flows across state lines, and in certain locations across international boundaries. Development and use of the ground-water resources in one state or nation may adversely affect development and use of the common resources on the other side of the state line or international boundary. Such interactions may run upstream as well as downstream.

In 1970, Bittinger (3) identified approximately 200 interstate and international aquifer situations throughout the coterminous United States. Of these, 30 were classified as facing major problems and 73 minor problems at that time. Nine major and 25 minor problems were expected to develop within 5 yr, i.e., by 1975. An additional 19 major and 42 minor problems were anticipated to develop in 5 or more yr. The types of problems identified included:

1. Excessive water-level lowering because of mining or low transmissibilities or both.
2. Reduction of surface water supplies due to pumping of ground water.
3. Encroachment of poor quality water because of heavy pumping of ground water.
4. Potential pollution of fresh ground-water supplies from waste disposal, not necessarily caused by development and use of ground water.

Despite the fact that these problems have progressively worsened since 1970 and additional problems have become evident, little has been done on a comprehensive, interstate basis.

Except for a management study now (1980–81) being conducted for the Delaware River Basin Commission under the writer's direction, the writer is not aware of any active interstate ground water resources management programs. The

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Delaware River Basin Commission and Susquehanna River Basin Commission, both formed pursuant to federal-interstate compacts (4,9), i.e., the United States is a signatory to the compacts, are the only interstate institutions with comprehensive ground-water resource management powers.

Nearly all of the actual experience to date in ground-water resources management has been at the intrastate level, by state agencies, or, more generally, by local agencies formed under state statutory authority. The legal and institutional arrangements for intrastate management vary widely among the states.

The major problems associated with developing and carrying out an interstate ground-water resources management program are in many respects the same as those involved with an intrastate program. The physical situation must be carefully assessed in both instances, the local people involved must understand the problem, believe in the need for action, and be willing to accept the constraints that may be necessary for effective management, and official government or other legal channels must be followed in order to gain the powers required to carry out the program. Having to deal with differing state laws and programs and the Congress make the legal and institutional arrangements is much more complicated for an interstate program than for an intrastate program.

Since the boundaries of the geologic formations which serve as aquifers and hydrology are not related to state boundaries, the physical problems are the same for interstate or intrastate aquifer systems. The same problems may develop when two neighboring communities draw from a common ground-water supply whether or not the communities are located in the same state. The range of interstate ground-water resource problems that have or will develop is as varied as the interstate aquifer systems that exist. The whole spectrum of problems, needs, and opportunities will be encountered on an interstate basis.

Often, the water users in the individual states see no advantage to be gained by management on an interstate basis and are resistant to imposition of any form of management by the state or federal governments. This paper emphasizes the legal, institutional, and attitudinal problems involved.

Two interstate aquifer systems, the Ogallala Aquifer, which underlies portions of eight High Plains states, and the Rio Grande Valley between Elephant Butte Reservoir in New Mexico and Fort Quitman in Texas, are examined briefly herein. The latter system is international as well as interstate.

Definitions

Ground Water Resources.—The term "ground-water resources" rather than "ground water" will be generally used in this paper. The term, as used here, includes not only the ground water itself but also the storage capacity in the underground formations that could be developed and utilized for a variety of purposes. The water-bearing subsurface formations, i.e., aquifers, may, in some cases, provide a means of conveyance of water to areas of need in place of pipelines or other aqueducts. Passage of water through the subsurface materials, particularly the near-surface soil column, may provide a degree of water treatment. In the general case, ground-water resources management encompasses the proper appraisal and effective use of all these subsurface attributes.

Aquifer System.—This term is used rather than the singular term "aquifer" or "ground-water basin" because, in the general case, two or more hydrologically

and hydraulically interrelated water-bearing formations are involved.

Ground Water Resources Management.—An adequate general definition of either "ground-water management" or "ground-water resources management" has not been found in the literature. Manual No. 40, "Ground Water Management," Committee on Ground Water, Irrigation and Drainage Division, American Society of Civil Engineers, 1972, perhaps the most comprehensive treatment of the subject to date, does not present a general definition, despite the fact that the term "ground-water management" has been used increasingly in water resource literature in recent years.

To many, particularly in the legal profession, management means simply reduction in extractions of ground water to bring total pumpage into balance with some value of the so-called "safe yield."

The California State Department of Water Resources in Bulletin 118-80, "Ground Water Basins in California," January, 1980, describes "ground-water basin management" in these terms:

> Ground water basin management includes planned use of the ground water basin yield, storage space, transmission capability, and water in storage. It includes (1) Protection of natural recharge and use of artificial recharge; (2) planned variation in amount and location of pumping over time; (3) use of ground water storage conjunctively with surface water from local and imported sources; and (4) protection and planned maintenance of ground water quality.

For this paper, the following definition has been adopted:

> Ground-water resources management involves the protection, development and use of the ground-water resources of the particular aquifer system concerned, generally in conjunction with available surface water resources possibly including reclaimed water, in the most effective manner to meet water resource development and use objectives as those objectives are defined and redefined with changes in conditions over time.

Ground-water resources management, as used herein, is not limited to the allocation of limited resources among competing users nor simply to control of extractions within the framework of some allocation scheme, although these may be part of the overall management program. Continued mining or overdraft is not precluded where justified in the public interest to support an existing economy. This might be the situation where an existing economy would be forced to significantly retrogress if extractions were reduced, resulting in severe economic, social, and possibly environmental disruptions. Rather, management is envisioned as a comprehensive package of legal, institutional, organizational, regulatory, and physical actions designed to protect the quality of the ground waters and to make best use of the ground-water resources conjunctively with surface-water resources for the benefit of those dependent on those resources. Because hydrologic and economic systems are dynamic, a management program must be periodically adjusted to conditions as they change over time.

Regulatory actions may involve control over the amount and areal pattern of extractions, control of well spacing, regulations governing the drilling, spacing,

and abandonment of wells, conservation regulations, control of waste disposal, and the like. Economic incentives may be used to control extractions and to achieve equity. Possible physical measures cover a wide range from, say, artificial recharge by surface spreading or injection to land treatment.

In formulating a management program, due consideration must be accorded to national, state, or regional interests in the development and utilization of the ground-water resources involved. Identifying and evaluating the scope and degree of those interests is difficult.

To conduct a successful ground-water resources management program, water users may be faced with the need to give up one or more of their avenues of totally independent action in order to achieve a common benefit. Experience has shown that individual users of ground water will not voluntarily conduct a management program. Users ordinarily and understandably produce water at the cheapest cost to them without much concern for the regional long-term advantages of conservation and management. It is difficult for a single user to be so concerned when he or she does not feel that his or her own operations are being jeopardized. This traditional attitude, however, may affect regional interests adversely because seldom does a negative effect on an aquifer system touch all users equally at the same time. These effects are progressive. Before all users are equally affected and equally concerned it may be too late to apply fully effective remedial measures.

If the users of water from a common aquifer system recognize the need for and are to support a management program, they must be willing to subject themselves to a management philosophy. This willingness may be the result of incentive or coercion. A workable incentive may be the equalization of the price of water irrespective of the cost and sources used.

The entire concept of ground-water resources management—from the standpoint of both quantity and quality—presupposes the availability of an adequate data base upon which to build a management program and administrative system. Generally, the data base must incorporate long-term, continuing detail regarding the geology, hydrology, economics, and environmental parameters, and the legal, institutional, and social structure of the area where management is being undertaken.

Present Legal Framework for Interstate Management

Regional administrative and management structures have been devised and are operational in dealing with interstate surface-water resources, regional economic development problems and activities, and other types of planning and management.

Existing Regional Models.—Regional models now existing for purposes that could be expanded to include comprehensive ground-water resources management, and some features of each, are:

1. Federal-interstate compacts. These compacts require Congressional consent, and include the federal government as a signatory party. Under this arrangement, certain federal powers and authorities for planning, construction, and operation of water resource projects are subject to the compact. Only two such regional models presently exist which provide the authorities required for comprehensive

ground-water resources management—the Delaware River Basin Commission and Susquehanna River Basin Commission.

2. Interstate compacts. In these interstate arrangements, the federal government is not a signatory party, although Congressional consent is required. Authority and enforcement powers are provided in the state enactments approving the compact arrangement. Compacts are in effect for many river systems in the United States, and in some cases are operationally active in managing the surface water resource allocations among the compact states.

3. River Basin Commissions. New England River Basins Commission and Missouri River Basin Commission are examples. These commissions were established under Title II, Water Resources Planning Act of 1965, Public Law 89-80. The commissions have coordination, planning, and program development responsibilities but lack authority to finance, construct, and operate, and have no enforcement powers. They must rely for success on intergovernmental and interstate coordination and cooperation and on consensus for decisions. The commissions have been active, in the areas where they have been formed, in basin-wide planning and program development. Ground-water resources are a part of the overall resource that should be considered in their broad planning role. Commissions have no authority, beyond providing a coordination mechanism, for enforcing compliance by state or federal agencies with commission-approved basin development plans and programs.

4. Regional federal agency. The Tennessee Valley Authority (TVA) model for this structure was created in response to compelling regional economic and resource problems during a period of innovative and unique federal response to such problems. It was crisis-generated, and has enjoyed considerable operational success. As formulated, however, local and state governments have no direct legal power in the Authority's formulation or execution of policies or programs.

5. Regional federal-state commissions. The Appalachian Regional Commission is an example. This institutional structure, like TVA, was created to meet a specific and geographically unique economic crisis. The Commission includes federal and state governments. Its enabling legislation provides Congressional authorization for federal and state agencies to plan and coordinate programs within the framework of Commission objectives.

Federal Statutes—Limited Management Powers.—Under many federal laws, particularly in the more recent environmental legislation, explicit and powerful federal authority is established which, if fully implemented, could impose federal controls, but not comprehensive management, in specific ground-water situations. Under present statutes, no federal agency has explicit, broad authority to engage in comprehensive ground water resources management. While EPA has certain broad authorities, as discussed below, these do not extend to the range of powers required for comprehensive management of an aquifer system.

1. Safe Drinking Water Act. Section 1424 (e) allows the Administrator (of EPA) "on his own initiative or upon petition" to make a determination that an aquifer is the sole or principal source of drinking water for an area. Based on that determination, administrators may control all federal assistance to projects that they find may contaminate the aquifer through its recharge zone. The

underground Injection Control projections of the act likewise accord the Administrator certain management powers. The potential for control under this act is enormous. In an area like the Ogallala Aquifer, where many towns and communities rely exclusively on the Aquifer for drinking water and where its recharge area extends over parts of eight states, the Administrator could, in effect, establish significant controls on ground-water resources use and, as a result, on economic development.

2. Clean Water Act (FWPCA, P.L. 92-500 as amended, Sections 102 and 208). While ground-water management is not explicitly legislated in P.L. 92-500, a degree of management authority and intent is implicit if ground-water quality is to be protected and maintained. The many interrelated sources and causes of ground-water pollution, and the inherent complexity of ground-water resource systems, make a comprehensive approach to management necessary for effective quality control.

Court Decisions.—There is little question that the federal government has constitutional authority to control the use of much of the surface-water resources of the United States. Court decisions appear to extend this authority to control of ground water, at least to interstate ground waters. The U.S. Supreme Court in *Arizona versus California,* 376 U.S. 340 (1964), dealt with the question of mainstream water extracted by "underground pumping," Article I(C). In Article IV(D), relating to interstate rights in the Gila River system in New Mexico and Arizona, "tributaries . . . and underground sources" of water are also expressly covered. While the Court has never adjudicated the rights to ground water not related to surface water in interstate equitable apportionment litigation, the fact is that in almost all cases ground and surface waters are interrelated.

In *United States versus Cappaert,* 426 U.S. 128, 142 (1976), the "pupfish" case, the U.S. Supreme Court took judicial notice of that relationship, and imposed restrictions on ground-water pumping for irrigation in Nevada to protect water levels at springs in Death Valley National Monument in California.

Professor Charles E. Corker (4), School of Law, University of Washington, believes that *People of Illinois versus Milwaukee,* 599 F.2d 151 (7th Cir. 1979), may provide the basis for litigation brought in the lower federal courts by one state against the people of an adjoining state with respect to management or lack thereof of common ground-water resources (personal communication, September 25, 1979).

Nonorganizational Institutional Factors.—A second approach to solution of ground-water resources management problems occurring across state lines would involve some combination of incentive and regulatory actions taken at the federal level, the regional level, by agreement among the states, or be self-imposed by water users. These might involve some modification or enhancement of existing laws or programs, or a new approach through new programs. To be successful, a major effort in public education would be required.

Ogallala Aquifer.—The Ogallala Aquifer underlies portions of eight High Plains states, as shown in Fig. 1. It was formed during early Pleistocene time by outwash from the ancestral Rocky Mountains. It is the principal source of water supply for some 20% of the irrigated acreage in the United States with over 15,000,000 acres irrigated in 1974. About 40% of the fed beef consumed in this country is fattened in the area.

The Aquifer is highly variable in saturated thickness and hydrologic and hydraulic characteristics. It is severely overdrawn throughout the area, although the degree of overdraft varies, being much less in Nebraska than in western Texas. Recharge is generally from precipitation and irrigation return flows plus some stream bed percolation, and varies widely over the area. Some portions of the Ogallala have already been dewatered, particularly in the southerly part of West Texas where the Aquifer is thin. The State of Texas has estimated

FIG. 1.—Ogallala Aquifer

that without a supplemental (imported) water supply about 40% of the present irrigated acreage in the High Plains of West Texas must revert to dryland farming or be abandoned by 2,000, and 60% by 2,020. The amount of water now remaining in underground storage varies widely. Nebraska, for example, has sufficient resources still to last for many years.

Except for Colorado, each of the states has developed a state water plan.

Massive intrastate, interbasin transfers have been envisioned for Nebraska, Oklahoma, and Texas.

The federal interest has so far been expressed through assistance programs, investigations, and some projects of localized extent and benefit.

No attempt has been made to manage the Ogallala Aquifer resources on a comprehensive, interstate basis even though the hydrologic, hydraulic, and resultant economic interdependencies across state lines are widely recognized. Limited intrastate management actions have been taken by local agencies formed under state statutory authority in Colorado, Kansas, Nebraska, and Texas.

Legal and Institutional Setting.—The laws of the several High Plains states concerning ground-water resources vary widely. State agencies in Texas have little or no authority to control or manage ground water. Rights to the use of ground water are based on the "rule of capture" as established by the Texas Supreme Court. Underground water conservation districts exercise a degree of management, largely through well spacing standards, enforcement of conservation measures, public information, and research. In New Mexico, the doctrine of prior appropriation governs the development and use of ground water. The State Engineer exercises exclusive administrative control over acquisition of rights to ground water. The Oklahoma Water Resources Board issues permits for use of ground water pursuant to certain statutorily prescribed criteria. The laws of the States of Colorado and Kansas respecting ground water are to a degree similar, requiring permits for appropriation of ground water with limitations on the amounts to be extracted with the objective of prolonging the life of available resources. Management authority is largely exercised through ground-water management districts. These districts, while having rather broad authorities, have been able to exercise their powers only to a limited degree, in part because of limited financial resources. Natural resource districts in Nebraska have broad powers over both ground and surface waters, but these, too, are hampered by financial limitations.

There is, of course, some economic competition among the states. Nebraska's objective is to irrigate every irrigable acre as rapidly as economic conditions provide justification.

Public Attitudes.—While the problems of the Ogallala Aquifer are widely recognized, there is widespread antipathy to any extension of state or federal control over the management of ground-water resources (2,6). There is currently little or no support for an interstate management institution and program. The ground-water users see no advantage to management in the absence of definite plans to increase water supplies.

Six-State High Plains-Ogallala Aquifer Regional Resources Study.—This $6,000,000 comprehensive resource and economic development study of the area in Colorado, Kansas, Nebraska, New Mexico, Oklahoma, and Texas, dependent on the Ogallala Aquifer and associated aquifers as the principal source of water supply, was authorized by Public Law 94-587. It is being conducted under the writer's direction for the Economic Development Administration, U.S. Department of Commerce. The study area comprises some 180 counties, or 225,000 sq miles (582,750 km^2) wholly or partly overlying the Aquifer.

Major emphasis is being given to improving the water supply situation by a variety of possible means ranging from local conservation through improved irrigation and agricultural management practices to major interstate, interbasin

transfers of surplus water from adjacent basins, the Missouri River, and streams in Arkansas. Effective management of the Ogallala ground-water resources,

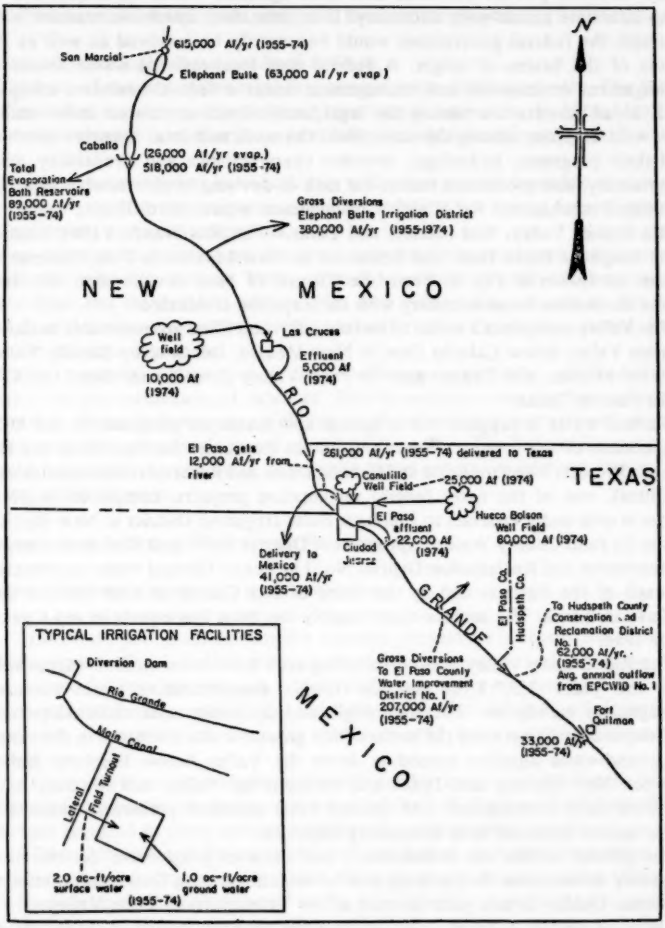

FIG. 2.—Diversions, Rio Grande Project [from Elephant Butte Reservoir—Fort Quitman Project—Preliminary by United States Department of the Interior, Bureau of Reclamation (Now Water and Power Resources Service)]

and legal and institutional mechanisms for comprehensive conjunctive management of ground- and surface-water resources are major elements of the study.

Administration of any program benefiting two or more of the states would

require a new type of management structure with broad powers, not only for planning but also for regulation, operation, and overall water resources management. Broader powers for state and local agencies would be required even if no interstate action were necessary. If an interstate, interbasin transfer were planned, the federal government would necessarily be involved as well as the states of the basins of origin. A federal-state-local-regional water resources management commission and management under a federal-interstate compact with broad powers are among the legal/institutional structures under study. The wide diversity among the state laws, the state and local agencies involved and their programs, hydrology, resource characteristics and availability, crop adoptability, and economics render the task of devising implementable legal/institutional mechanisms for effective management especially difficult.

Rio Grande Valley, New Mexico, and Texas.—The Rio Grande Valley extends from Elephant Butte Dam and Reservoir in New Mexico to Fort Quitman in Texas, as shown in Fig. 2. From the City of El Paso downstream, the river forms the international boundary with the Republic of Mexico.

The Valley comprises a series of incised, alluvium filled ground-water basins—Rincon Valley below Caballo Dam in New Mexico, followed by Mesilla Valley in New Mexico and Texas, and El Paso Valley downstream from the City of El Paso in Texas.

Surface water is supplied for irrigation and municipal purposes by the U.S. Department of the Interior, Water and Power Resources Service, from the Rio Grande Project (Elephant Butte and Caballo Dams and Reservoirs and appurtenant facilities), one of the early federal reclamation projects, completed in 1915. Water is sold under contract to Elephant Butte Irrigation District in New Mexico and to El Paso County Water Improvement District No. 1 and Hudspeth County Conservation and Reclamation District No. 1 in Texas. Ground water is extracted by each of the districts and by the cities of Las Cruces in New Mexico and El Paso in Texas. The surface-water supply has been inadequate in most years since 1951.

The Rio Grande Valley and surrounding area have been studied extensively over the years (1,5,7,8,10) but not from a comprehensive water resource management standpoint. The hydrologic and hydraulic interrelationships and interdependencies between the surface- and ground-water resources in the series of ground-water aquifers extending down the Valley below Elephant Butte, between New Mexico and Texas and between the Valley and Mexico, have not been fully investigated. The ground-water resource problems, needs and opportunities have not been adequately explored.

The ground waters are overdrafted, and there is progressive deterioration in quality downstream due in large part to irrigation return flows and municipal effluents. Quality is now poor in most of the Texas portion of the Valley.

There has been no attempt, even little thought, given to ground-water resources management on an intrastate, interstate, or international basis. The International Boundary and Water Commission has given some consideration to the international problems.

Legal and Institutional Setting.—The allocation and use of the surface waters of the Rio Grande below Elephant Butte are controlled by:

1. The treaty with Mexico.

2. The Rio Grande Compact among the States of Colorado, New Mexico, and Texas.
3. Court decisions.
4. The water sales contracts under the Rio Grande Project.

Ground-water rights have not been defined.

The laws of the two states regarding ground water differ markedly. The State of Texas has no direct control over ground-water resources. In New Mexico, the state engineer is vested with responsibility for administration of ground-water rights.

The three water districts have authority to develop ground water but no powers to manage ground-water resources. El Paso, while it is a heavy user of ground water, has no management powers.

Any attempt to institute an interstate ground-water resources management program in the Rio Grande Valley would be extremely difficult because of these legal and institutional complexities and differences, as well as the fact that New Mexico, the upstream state, is much more favorably situated from a water resource standpoint.

Public Attitudes.—Here, too, there is widespread antipathy, particularly in Texas, to any extension of state or federal control over the ground-water resources. There is, however, a high degree of national and international interest in the water resources, surface and ground, of the valley. The Federal Government would necessarily be involved, at least to some extent, in any comprehensive management program.

Developing an Interstate Management Program

The following general aspects are relevant:

1. Interstate and intrastate aquifer systems present no inherent management distinctions insofar as their physical characteristics are concerned. The potentials for successful management are widely different, however, because the tools of management—data availability, statutory authorities, institutional design and continuity, user attitudes, financial resources, and many more—differ across state lines.
2. Ground-water user attitudes and attitudes for those representing users are of key importance in developing and accomplishing the legal and/or institutional changes required to carry out a comprehensive management program.
3. Two factors must be established if acceptance and implementation of a comprehensive ground-water resource management program is to be achieved: (1) Users who would be subjected to the program must perceive that a problem of sufficient gravity to threaten their individual well-being exists; and (2) the benefits to be achieved by a management program outweigh the loss of individual choice. In an interstate case, the difficulty of establishing both these factors is compounded. Without full recognition and understanding, and willingness, management would be politically and practically impossible.
4. In the United States, the only governmental authorities that could operate without restraint across state lines are: (1) The Federal Government; and (2) those agencies created by interstate compact. It seems clear that the Federal

Government has power to establish an interstate authority to manage ground-water resources, or to create a mechanism by which this could be done. The Federal Government by itself has not yet done so. The need for solutions that are specifically adapted to particular regions and localities makes it unlikely that Congress will soon do so. The first step should be to support efforts by the states, municipalities, water districts, or other local authorities.

5. Generally, management can be most effectively accomplished at the local or regional governmental level, operating within a framework of powers and duties established by state statutes and, for interstate aquifer systems, under broad federal authority.

6. Current laws, both statute and case, concerning water rights impede and, in some cases, block effective management. Principal weaknesses in the present legal and institutional posture at the state level stems from these basic points: (1) In some states, private ownership of ground water attaches through ownership of the land surface, and the states have not enunciated or implemented jurisdiction in terms of allocation or administration of the resource; and (2) state law and court decisions have generally dealt with surface and ground water as separable resources.

7. The set of physical problems and boundaries existing for each aquifer system uniquely prescribes the set of physical ground-water resources management activities that can be applied to it. The economy that exists provides additional, although equally important, constraints about the management solutions available. Therefore, any guidelines about ground-water resources management that would apply to more than one aquifer system must be general in nature. A specific set of guidelines for ground-water resources management cannot be applied uniformly throughout the country. Because there are major physical, economic, environmental, and social differences between aquifer systems and the uses thereof, *each ground-water resource situation requires a unique management program.*

8. Continuing dissemination of information to the public and public participation in planning and decision-making are essential.

9. A management program to be successful must be sufficiently flexible to adapt readily to changing hydrologic, economic, institutional, environmental, and social conditions.

When the ramifications of these aspects are considered, it follows that the legal and institutional factors that must be considered in formulating a ground-water resource management structure are largely dictated by the social, environmental, and economic goals of the area involved. Implementation of such management requires that those goals be articulated, management tools to allocate the total water resources equitably among users be available, and government actions required for management be initiated and carried forward.

The objectives sought through management of ground- and surface-water resources on a conjunctive "systems" basis are not the same from area to area. Objectives that might be important in one area, such as extending the duration of availability of water from the aquifer system, maintaining spring flows and stream base flows or controlling subsidence might have little relevance elsewhere.

Many alternative institutional structures could be considered for the manage-

ment vehicle in the broad context described above. The extremely diverse hydrologic, geologic, economic, environmental, legal, political, and social conditions affecting the occurrence, protection, and use of ground- and surface-waters in the United States suggest that no single structure would be universally applicable nor politically acceptable.

In the general case, assuming that there is one or more entities available to initiate action, the following guidelines, more or less in sequence, are proposed as an implementable process to develop and implement a management program for an interstate aquifer system:

1. Define and delimit the aquifer systems and ground-water resources involved, generally based largely on available data and information.
2. Define the federal and individual state and local interests, present and prospective.
3. Initiate and carry out any additional investigations necessary to provide an adequate data base—geology, hydrology, other sources of water supply, economics, environmental, legal, institutional, and social.
4. Define the ground-water resource problems, needs and opportunities, present and prospective.
5. Define the area to be encompassed within the management program. Generally, this should be based mainly on geologic and hydrologic considerations, although for institutional purposes, it may be necessary to consider political boundaries as well. Those aquifers with significant hydrologic and hydraulic interdependencies should be included.
6. Define the management objectives.
7. Define the physical actions and the type and extent of controls that may be needed to solve identified problems, and to make effective use of the available resources in fulfillment of the objectives.
8. Define the legal, institutional, and organizational structure necessary for management.
9. Institute a public information and participation program.
10. Secure the requisite legislative enactments.
11. Establish the institutional and organizational structures for management.
12. Procure necessary professional, technical, legal, and administrative staff and expert assistance.
13. Develop the management plan maintaining flexibility to adapt to changing conditions.
14. Develop the management program to implement the plan.
15. Arrange for any financing required for construction of necessary facilities.
16. Set tax rates, fees, and other charges necessary.
17. Implement the program.
18. Continue data collection, monitoring, investigations, and planning.
19. Amend the objectives and plan and revise the program as necessary from time to time to adapt to changed conditions.

Conclusions

No interstate agencies presently have powers to manage interstate ground-water resources comprehensively except the Delaware River Basin Commission and

the Susquehanna River Basin Commission, both formed under federal-interstate compacts.

Initiation of Action.—The first, necessary step of initiating action may be the most difficult of all in the development, adoption, and implementation of an interstate ground-water resources management program—far more so in an interstate situation than intrastate. In most instances achieving any degree of consensus among the states and water users involved will involve major difficulties. In the case of the Delaware River Basin federal-interstate compact, active citizen associations took the initiative, but this appears not to be the general case. In that situation, the preponderance of population, economic development, ground-water usage, and problems occur downstream, and three of the four states involved in the compact are adversely affected by the ground-water resources problems.

Several factors militate against interstate agreement to initiate action leading to an interstate ground-water resources management program, including but not limited to:

1. Water users in one or more states, generally upstream, may be in a much more favorable position with respect to the ground-water and other resources and, thus, be reluctant to be restricted in any way for the benefit of those downstream.
2. Conflicts and competition among states.
3. Diversity of the water laws of the states.
4. Differing bases for ground-water rights.
5. Lack of information and data concerning the ground-water resources, and problems, needs, and opportunities.
6. Lack of understanding of the ground-water resource problems and opportunities, and of the interstate hydrologic and hydraulic interrelationships.
7. Lack of definition of the interstate problem(s).
8. Lack of understanding of any advantages that might result from an interstate management program.
9. Resistance by many water users to any sort of imposed management.

Legal action brought in federal or state courts by an aggrieved state or states against the other state or states involved with an interstate aquifer system could be the "trigger" to force the states and water users to get together on an interstate management program. This has resulted on a system-wide basis from some of the intrastate adjudicatory actions brought by one or more aggrieved water users in California.

Legal and Institutional Management Structure.—Some possibilities, interstate compacts and agreement among the states, would require lengthy and complicated negotiations. In the meantime, ground-water problems might become much more serious, perhaps to the extent of loss of the ground-water resources through quality deterioration, as in portions of the Rio Grande Valley below Elephant Butte, or through exhaustion as will be the result in portions, at least, of the Ogallala Aquifer unless remedial actions are taken in the near future.

National, federal, state, and local interests and responsibilities must be recognized and incorporated in any comprehensive interstate ground-water resources management program if it is to be implementable. To this end, the

writer believes that the management program might well be formulated and implemented under the direction of a regional water resources management commission established pursuant to a federal statutory enactment with representatives of the states and principal local agencies involved but without federal domination. Such a regional commission would be broader in scope than any existing federal regional or river basin commission because it would be an implementing or action agency as well as a planning agency. It would need all the powers and authorities necessary to carry out the management program. However, implementation should be by and through the local and state agencies to the maximum extent possible. This probably would require state legislation to broaden the powers of local agencies and, perhaps, the state agencies.

The organization and powers of a regional water resources management commission would necessarily be tailored to the specific interstate situation involved. Creation of such a commission by federal enactment would require extensive discussions with the states and local agencies involved; and the concurrence of most, if not all, to be politically viable.

The writer believes that further study should be given to this concept of a regional water resource management commission for interstate aquifer systems.

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DISCUSSION

Note.—This paper is part of the Journal of the Water Resources Planning and Management Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. WR2, October, 1981. ISSN 0145-0743/81/0002-0581/$01.00.

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Two Centuries of Water Planning Methodology[a]

Closure by Ray K. Linsley,[3] F. ASCE

The author appreciates Armstrong's interest in this paper and his comments which stem from many years of experience.

All of the definitions of planning given in the paper have been used in the past and some are still in use. In an appropriate context, most of them could be considered to be valid definitions of planning. They were mentioned in the paper to point out one of the difficulties of analyzing the evolution of planning methodology over two centuries, and not to cast innuendoes at the civil engineering profession.

The author's statement, that formal consideration of environmental factors in planning is relatively new, is based on the same interpretation of the word environment as used by Congress in enacting the National Environmental Protection Act in 1969. The fact that Congress felt it necessary to produce such an act indicates that, in their view at least, prior planning had been less than adequate, both with respect to environmental considerations and to public participation in the planning process.

Polling any large group of people on a specific issue will usually elicit a wide spectrum of opinions. Professions do not necessarily represent uniformity of opinion on all issues. Among engineers one might expect to find some unanimity on technical issues, but even on such issues there are often differences of opinion. On subjective issues, such as the "environment," wide differences of opinion exist within the profession. In the end, however, the opinions of the public, which we serve, must control planning decisions on subjective issues. The requirements of environmental impact analysis imposed by legislation and administrative regulation are often burdensome and sometimes unnecessary. To escape from these burdens, the profession must propose a substitute procedure which satisfies all parties concerned.

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[a]March, 1979, by Ray K. Linsley (Proc. Paper 14405).

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MAP SKEW???[a]

Closure by Richard H. McCuen,[3] A. M. ASCE

Skew is an important input parameter to flood frequency analyses. Recognizing that estimated peak discharges are very sensitive to values of skew, it is important to have accurate estimates. Because estimates that are based on short records are less likely to be accurate, efforts are made to use regional information to adjust estimates of skew that are made from on-site streamflow data. The skew map and the regression equation are two methods of regionalizing streamflow statistics.

Before commenting on the discussion paper, it should be emphasized that the map provided in the paper (Fig. 2) was not meant to be used in place of Plate I of Ref. 2. Fig. 2 was presented ". . . to show that Plate I is not unique and that there are probably a multitude of maps that could be developed with overall accuracy similar to the accuracy of Plate I." The accuracy of Fig. 2 was not significantly better than the accuracy of Plate I, although the standard error of Fig. 2 was slightly lower.

The members of the Committee that developed Plate I were aware that the skew map would be sensitive to the censoring or adjustment of outliers, both low and high. However, since standardized methods were not available and widely accepted by the professional community, a decision was made to minimize data modification. Thus, the measurement data points that were identified by the discussor were not adjusted.

The discussor indicated that he "developed and computerized a simple methodology for modifying any perceived outliers so that they fit the trend of the rest of the data." However, the discussor failed to provide either the details or a reference so that readers would be able to judge the validity of his findings. Removal of low outliers would obviously increase the skew, decrease the standard deviation, and increase both the mean and Q_{100}. A major consideration in judging the value of the discussion paper centers on the acceptance of the practice of modifying measured data. I am totally unaware of any hydrologist or engineer ever recommending changing the value of a flood peak because one considers it an outlier. All previous procedures, including Bulletin 17a, recommend either censoring or adjusting the probability plotting position; but to modify a discharge rate that was measured with a streamgage does not seem rational. Hydrologists recognize that the computed plotting positions may not be the true value and so methods of adjustment have always been based on the adjustment of the plotting position.

Finally, the discussor states, "A preliminary analysis indicates that when outliers are adjusted, the distribution parameters indicate correlation with basin and stream factors." The analysis is not provided for the reader's consideration

[a]September, 1979, by Richard H. McCuen (Proc. Paper 14840).

[3]Professor, Dept. of Civ. Engrg., Univ. of Maryland, College Park, Md. 20742.

and so there is no way to judge the validity of the statement. I hope the analysis was based on more than the six stations used by the discussor in his other analysis; after all, a correlation analysis with only six observations can be very misleading.

The author appreciates the efforts of the discussor. Given the importance of flood frequency analysis and the minimal amount of research that is currently being performed, it is encouraging to know that the discussor is involved in research on the subject.

Ground-Water Management with Fixed Charges[a]

Discussion by K. Elango[3]

The authors, who pioneered the formulation of finite difference based linear programming models for aquifer management are to be congratulated for the further improvement proposed by them by way of inclusion of fixed installation charges of wells, thus leading to an economic objective function, as against an engineering or physical objective function. It is noted that, in the present case, the optimization framework gets extended as a mixed integer programming formulation.

The writer is working on the development of a multistage decision model for a mine-water depressurization well points system, that has to keep pace with a continuously shifting area of mining activity. Therein arises the question of the economically optimal *life-time* of a well, for deciding the abandonment of the least efficient well and the installation of a new well near the active mine area. This interest prompted the writer to look into the relative significance of the fixed installation charges with respect to the time horizon of operation of wells, in the authors' study.

The presentation and discussion of the results of the sample problem in the paper, are limited to the comparison of only the engineering component of the optimal solutions for the two cases, namely the one without and the one with the assessment of the fixed costs of wells. Since the area of a finite difference-computational cell is 400 m^2, the minimum cost solutions, z^*, in dollars, can be written in terms of the number of days of pumping, t as

$$z^* = 3{,}295\, t, \quad \text{when fixed charges are not considered} \quad \ldots\ldots\ldots\ldots (6)$$

$$\text{and} \quad z^* = 3{,}589\, t + 52{,}320 \quad \text{when fixed charges are considered} \ldots\ldots\ldots (7)$$

Table 2 presents a comparison of optimal costs for various pumping time horizons (discarding the discounting aspect). The ratios of the optimal costs indicate that even for a time horizon of two years the neglect of the fixed charges

[a]July, 1980, by Eduardo Aguado and Irwin Remson (Proc. Paper 15521).

[3]Assoc. Prof., Dept. of Civ. Engrg., Indian Inst. of Tech., Madras, India.

would underestimate the optimal costs by about 11% for the adopted values of cost coefficients of the sample problem, while the asymptotic value of this ratio works out as 1.09.

The writer wishes to know what time horizon was implied in the study of the authors. To what extent would these results be vitiated by the hydraulic transients in a newly initiated dewatering system? Is it worthwhile to treat the coefficients c_{ij} to be functions of local hydraulic heads and pumpages? If so, can the mathematical programming package be used as an inner segment within an iterative FORTRAN program, so that the results from the package be automatically used for the next iteration? Lastly the writer wishes to know whether there was any particular difficulty encountered in obtaining solutions, due to the presence of a relatively large number of constraints of the equality

TABLE 2.—Optimal Costs to Maintain Low Water Levels in Excavation without and with Assessment of Fixed Installation Charges and their Ratio as Function of Pumping Period

Number of days of pumping (1)	Optimal Cost in Dollars		Ratio of optimal costs = (3)/(2) (4)
	Without fixed charges (2)	With fixed charges (3)	
1	3,295	55,909	16.97
7	22,813	77,443	3.39
30	98,850	159,990	1.62
60	197,700	267,660	1.35
90	296,550	375,330	1.27
180	593,100	698,340	1.18
365	1,202,675	1,362,305	1.13
730	2,405,350	2,672,290	1.11

type, arising out of the flow-equilibrium equations. The writer's experience in this regard reported elsewhere (15) has prompted this question.

Appendix.—Reference

15. Elango, K., Suresh Rao, H., and Indrasenan, N., "Linear Programming Formulation with Predominance of Equality Constraints," *Proceedings of the 15th Annual Convention of Computer Society of India*, Bombay, India, Vol. 3, Jan., 1980, pp. 96–101.

Discussion by G. Padmanabhan[4]

The authors have formulated the dewatering problem as a mixed integer linear programming formulation by including both installation and pumping costs of the wells. Earlier, the dewatering problem was formulated as a continuous LP formulation without including fixed installation costs, keeping the minimization of total pumping as the objective function (2). The optimal solutions (pumping

[4]Asst. Prof., Dept. of Civ. Engrg., North Dakota State Univ., Fargo, N.Dak.

schedules and well distributions) from both the LP formulations have been compared by the authors.

Although the finite difference and finite element approximations to the governing equations of ground water flow have been shown to lend themselves for LP formulation, an important aspect of LP formulations, the existence and significance of alternate optimal solutions have not been discussed by many. A problem in ground-water management or in general water resources management may have several alternate optimal solutions. For successful management of aquifer, it is also necessary to explore the possibility of existence and the usefulness of alternate optimal solutions. Final choice among them may depend on several factors other than the one characterized by the objective function. This may particularly be so when one has choice on the production wells in a pumping and transportation scheme. Though it is well known that the optimal solution given by many commercial LP packages may not be unique, not many have attempted to seek and present the alternate optimal solutions. A procedure developed by S. Sadagopan is capable of generating at least a few of the alternate extreme point optimal solutions for an LP. Though not exhaustive, they do provide additional possibilities for the management.

Three examples from Ref. 1 are shown herein using Sadagopan's procedure to have at least one alternate optimal solution each. Complete problem description may be had from the original reference. For brevity only the solutions are presented in all cases.

1. The LP problem for the case of a one-dimensional confined aquifer with fixed head boundaries at both ends under steady state conditions is chosen as the first example. Using the same numerical values as in the paper (1) for $m = 500$, the code indicated one alternate optimal solution as shown in Table 3 ($L = 40$ ft; $\Delta x = 10$ ft; $T = 10{,}000$ sq ft/day; $h_0 = 100$ ft; $h_4 = 110$ ft; $m = 500$ ft/day).
2. The case for the steady-state two-dimensional confined aquifer with fixed head boundaries is chosen next for the required total productions of 0.04 and 0.4 ft/day ($m = 0.01$ and $m = 0.10$). The solutions are presented in Table 4 ($T = 10{,}000$ sq ft/day; $\Delta x = 1{,}000$ ft; $h_p = 20$ ft for $p = 2, 3, 5, 8, 9, 12, 13, 16, 17, 20, 22, 23$).
3. The case of transient one-dimensional linear example of confined aquifer

TABLE 3.—Optimal Solutions for Example 1

Variable (1)	Solution presented in Ref. 1 (2)	Alternate optimal solution (3)
h_1	101.25	100.00
h_2	102.50	102.50
h_3	103.75	105.00
W_1	0.00	250.00
W_2	0.00	0.00
W_3	500.00	250.00
Z(obj. function value)	307.50	307.50

with fixed head boundaries is taken next (Table 5). The code generated the following solutions: $L = 40$ ft; $\Delta x = 10$ ft; $\Delta t = 0.0001$ day; $T = 10{,}000$ sq ft/day; $h_{1,0} = 110$ ft; $h_{2,0} = 120$ ft; $h_{3,0} = 130$ ft; $W_{1,0} = W_{2,0} = W_{3,0} = 0$ ft/day; $m_1 = 150$ ft/day; $m_2 = 100$ ft/day.

In example 1, while the solution presented in the paper (1) indicated W_3

TABLE 4.—Optimal Solutions for Example 2

Variable (1)	$m = 0.1$		$m = 0.01$	
	Solution in Ref. 1 (2)	Alternate optimal solution (3)	Solution Ref. 1 (4)	Alternate optimal solution (5)
W_6	0.1	0.1	0.01	0.01
W_7	0.1	0.1	0.01	0.01
W_{10}	0.0	0.0	0.00	0.00
W_{11}	0.1	0.1	0.01	0.01
W_{14}	0.0	0.0	0.00	0.00
W_{15}	0.0	0.0	0.00	0.00
W_{18}	0.0	0.0	0.00	0.01
W_{19}	0.0	0.1	0.01	0.00
h_6	15.57	15.59	19.56	19.56
h_7	15.15	15.14	19.51	19.52
h_{10}	17.13	17.23	19.50	19.73
h_{11}	15.05	14.96	19.72	19.50
h_{14}	17.92	18.35	19.84	19.74
h_{15}	17.90	17.46	19.75	19.81
h_{18}	16.64	18.72	19.87	19.67
h_{19}	18.63	16.54	19.65	19.87
z(obj. function value)	133.99	133.99	157.40	157.40

TABLE 5.—Optimal Solutions for Example 3

Variable (1)	Solutions in Ref. 1 (2)	Alternate optimal solution (3)
h_{11}	109.08	109.08
h_{21}	119.47	119.47
h_{31}	129.76	129.76
W_{11}	150.00	150.00
W_{21}	0.00	0.00
W_{31}	0.00	0.00
h_{12}	109.07	109.52
h_{22}	119.32	119.32
h_{32}	129.65	129.20
W_{12}	100.00	0.00
W_{22}	0.00	0.00
W_{32}	0.00	100.00
$Z = h_{12} + h_{22} + h_{32}$	358.04	358.04

to be the only pumping well, Sadagopan's code generated an alternate solution in which W_1 and W_3 were found to be pumping wells. The change of pumping wells can be exploited advantageously depending on the water requirements in the immediate vicinity of the corresponding wells. This is the same case with example 2. W_{18} and W_{19} are interchanged as pumping wells in the two solutions for both the cases of $m = 0.1$ and $m = 0.01$. Even intuitively, this was evident from the pattern of disposition of wells and the boundary conditions. In example 3, the alternate optimal solution has the production of the permissible lower limits of the first and second time steps from node 1 and node 3 respectively, whereas in the solution reported in the reference it was always from node 1. Correspondingly there is a change in the head distribution too.

Until an algorithm to generate all the extreme point alternate optimal solutions is developed it is but proper to make available to the Management at least a few of the alternate optimal solutions to the extent possible and leaving the final choice to the decision maker. Although the examples indicated are for the continuous LP formulation of ground-water flow to wells, the possible existence of such alternate optimal solutions is common to all LP formulations and thus must be explored.

Appendix.—Reference

15. Sadagopan, S., "On Ranking the Extreme Points of Convex Polyhedra," unpublished Master's Thesis, School of Industrial Engineering, Purdue University, Dec., 1977.

Chicago Region's Valuable Water Resources[a]

Discussion by Herbert Moore[2]

The author has set forth how the Chicago Region's valuable water resources have been protected by enlarging the channel of the Chicago River Waterway to divert Lake Michigan water to the Mississippi watershed, thus affording protection and fulfilling needs for navigation and water pollution abatement by dilution.

However, water pollution abatement by dilution is adversely dealt with by Public Law 92-500 which, in 1972, declared that "it is the national goal that the discharge of pollutants into the navigable waters to eliminated by 1985."

Accordingly the challenge to Chicago is to reduce diversion to that required for navigation and provide for sewage disposal to meet the zero discharge goal.

Such a program would entail collaboration with the State to augment the

[a]July, 1980, by Clint J. Keifer (Prof. Paper 15569).

[2]Consulting Engr., Moore & Peterson, Engineers, 2351 North Humboldt Boulevard, Milwaukee, Wisc. 53212.

study by the Corps of Engineers, published in the Journal of the Environmental Engineering Division in October 1973 with discussion by the writer in 1974. Studies should include: deep tunnel aeration and pre-treatment, land disposal augmenting aquifer recharge, and power generation through pumped storage.

Reducing Hydrologic Parameter Uncertainty[a]

Errata

The following corrections should be made to the original paper:

Page 247, Eq. (4): Should read

"$\hat{G} = W_G G_s + (1 - W_G) G_r$"

instead of

"$\hat{G} = W_G [G_s + (1 - W_G)] G_r$"

Page 248, Eq. (7): Should read

"$\hat{\theta} = W_\theta \theta_s + (1 - W_\theta)\theta_r$"

instead of

"$\hat{\theta} = W_\theta [\theta_s + (1 - W_\theta)] \theta_r$'

Page 248, Eq. (8): Should read

"$\text{Var}(\hat{\theta}) = W_\theta^2 \text{Var}(\theta_s) + (1 - W_\theta)^2 \text{Var}(\theta_r)$"

instead of

"$\text{Var}(\hat{\theta}) = W_\theta^2 [\text{Var}(\theta_s) + (1 - W_\theta)^2] \text{Var}(\theta_r)$"

Page 254, Eq. (26) should be renumbered and: Should read

"$\hat{Y} = W_{\bar{Y}} \bar{Y}_s + (1 - W_{\bar{Y}}) \bar{Y}_r$ . . . . . (28)"

instead of

$\hat{Y} = W_{\bar{Y}} [\bar{Y}_s + (1 - W_{\bar{Y}})] \bar{Y}_r$ . . . . . (26)

Page 254
Eq. (27) should be renumbered (29)
Page 254, Eq. (28) should be renumbered and: Should read

"$\hat{S} = W_s S_s + (1 - W_s) S_r$ . . . . . (30)"

instead of

[a]March, 1981, by Yeou-Koung Tung and Larry W. Mays (Proc. Paper 16136).

$$\hat{S} = W_s [S_s + (1 - W_s)] S_r \quad \ldots \quad (28)$$

Page 254
Eq. (29) should be renumbered (31)
Eq. (30) should be renumbered (32)
Page 255
Eq. (31) should be renumbered (33)
Eq. (32) should be renumbered (34)
Eq. (33) should be renumbered (35)
Last paragraph, line 6, "(Eq. 31)" should read "(Eq. 33)" and line 7, "Eq. 33" should read "Eq. 35"
Page 257
Eq. (34) should be renumbered (36)
Paragraph 2, line 5, "Eq. 33" should read "Eq. 35" and "Eq. 34" should read "Eq. 36"

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